A PARAMETRIC STUDY OF FLOOD ROUTING THROUGH A RIVER

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By

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CERTIFICATE

The thesis 'A PARAMETRIC STUDY OF FLOOD ROUTING THROUGH A RIVER' by Vinay Kumar is hereby approved as a creditable report on research carried out and presented in a manner which warrants its acceptance as a prerequisite for the degree of MASTER OF TECHNOLOGY. The work has been carried out under my supervision and has not been submitted else-where for a degree.

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SYNOPSIS

A knowledge of the way in which a flood wave attenuates as it travels downstream along a long open channel with a continuously moving unsteady- steady boundary is of importance in flood routing.

Implicit scheme was used and a dimensional analysis was carried out to consider all the significant parameters that affect the flood propagation. A systematic analysis was made by varying all the parameters in the practically possible range.

It was found that relative flood wave amplitude depends upon initial wave amplitude ratio of time to peak to time to centre of gravity of the inflow hydrograph, and Manning's n. The first two terms take into account the rate of rise of hydrograph, time of travel of the wave peak is chiefly dependent upon initial wave amplitude. These conclusions are very important in prediction of the peak stage and time of its occurrence.

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LIST OF SYMBOLS

A	=	Area
C	=	Intercept on ordinate on XY plane
C_o	=	Celerity of wave $=\sqrt{g \text{ x depth of flow}}$
g	=	Gravitational Constant
m	=	Slope of the time on XY plane
n	=	Manning's roughness
q_o	=	Steady flow discharge
Q_*	=	Relative discharge rise
t_p	=	time to peak of the inflow hydrograph
t_g	=	time to centre of gravity of the inflow hydrograph
T_p	=	Time to peak since starting of the hydrograph
T_*	=	Dimensionless time to peak
V_o	=	Initial steady state velocity
X	=	Dimensionless distance $= \frac{x}{y_o}$
x	=	Distance along the channel
y_o	=	Steady flow depth
y_w	=	Wave amplitude
Y_w	=	Dimensionless wave amplitude $= \frac{y_w}{y_o}$
Y^+	=	$1 - Y_*$
Y_*	=	Relative wave amplitude

LIST OF FITURES

CHAPTER I

INTRODUCTION

Our civilization has grown on the banks of rivers and for hundreds of years they were and are still a boon to mankind. But with the development of science, man tried to bind these rivers to flow on a particular path, wanted to cross them through strong bridges and thus to a great extent disturbed their natural flow. Some times dams are constructed, water is regulated in a particular way and thus taking the control in one's hand. One of the most important of all the unsteady flow phenomena the engineer has to deal with, is the movement of a flood wave down a channel, usually in the case of a river, and the problem along with this is the tracing of this movement and any related changes in the form and height of the wave. It is necessary that the engineer should possess theoretical or accurate semi-empirical means of determining the behaviour of a flood wave in a channel of specified form and slope, so that he may be able to predict the effect of the changes which he often makes in a channel in the interests of channel improvement and flood control.

Flood routing is the process of tracing by calculation the course of a flood wave and in large measure this problem is an application of the principles of unsteady flow. However a flood wave has its own features and may require its own special techniques for exploring these features. As for example, rise and fall of a flood occurs much more slowly than many of the flow changes in unsteady flow phenomena and so some of the acceleration terms in the equation of motion may not be of much significance, or otherwise, when the flood wave propagates, it becomes longer and lower as it moves downstream and this subsidence is very important for a practicising engineer. He may, therefore prefer instead of a general solution, an approach which concentrates on those aspects of the solution that are particularly relevant to the subsidence problem.

A semi-empirical method known as the Muskingum method, first developed by McCarthy [31] has been in use for many years. This method, which uses the equation of continuity and some empirical constants which characterise the basin concerned was presented by

Linsley [20]. This coefficient method is by no means an exact method of routing hydrographs in as much as only a few of terms of governing equations are considered in the derivation. The method though adequate for many purposes such as in the planning and designing stages of flood control or multiple-purpose projects, reduction in natural flows resulting from the control of floods by reservoirs etc., is of limited utility as it does not take into account the dynamics of flow. In recent years the advancement of computer technology has estimulated the development and application of a more rigorous approach of the numerical integration of the equations of motion and continuity for unsteady flow. A great deal of effort has been made to develop computational methods applicable to the routing of a flood wave in a complex natural or hypothetical drainage system. However, it is worth mentioning that the significant factors influencing the unsteady flow phenomena are so numerous and any typical watershed is so complicated that it is very difficult to arrive at any reliable conclusion.

The precise role of each factor affecting the flood wave propagation in open channels can best be understood by studying the flow in a relatively simple and ideal channel in which all important parameters can be independently and systematically varied. For example, the relations among important flood characteristics, such as maximum stage with Manning's roughness, initial wave amplitudes, time to peak of the hydrograph, distance etc. may be studied. It may be also possible to arrive at a result which can be used to get an empirical formula and thus help in flood prediction.

An approach of this type in open channel was applied by Mozayaney and Song. But their work is of very restrictive nature as the wave amplitude and, the length of the channel are small.

The transient flows due to a +vely skewed hydrograph of given duration at the upstream end of long rectangular straight channel were computed by implicit method. Implicit method was used because it is a very stable method and can be used very easily in

the case of natural rivers as shown by Amein in his paper [4]. Though the stability of the scheme makes its use convenient and easy for various dT and dx i.e. time step and distance step, it will have some numerical damping.

The inflow hydrograph taken was **+vely** skewed one because most of the practical hydrographs resemble this shape. Sine curve can also be used but then it does not represent the actual field problem and since the motive is to study subsidence with more practical input data, log pearson type III hydrograph was considered more useful. The initial wave emplitudes were 0.5, 1 and 2 times the initial water depth, the ratio of time to peak to the time to center of gravity of the hydrograph was 1/3 and 2/3, Manning's roughness coefficients were 0.02, 0.025, 0.03. These parameters were used in different groups and the resulting propagation of the flood wave was calculated. The damping of the peak stage and the time of occurrence of peak were studied in detail which will be presented in this report.

CHAPTER II

GOVERNING EQUATIONS AND SOLUTION

2.1 Assumptions

The one dimensional equations of continuity and momentum of unsteady flow in open channels are referred to as St. Venant's equations. These equations are derived in any standard text book on unsteady flow e.g. Mahmood and Yevjevich, Chow, Henderson etc. The following are important assumptions in the derivation of these equations.

(1) The channel is straight and uniform in the reach so that the one dimensional approach can be applied.

(2) Velocity distribution across the wetted area does not substantially affect the wave propagation.

(3) Friction losses are given by Manning's formula i.e. it is equal to loss in steady flow conditions.

(4) The average slope of the channel bottom is small therefore $\tan \alpha = \sin \alpha$ and $\cos \alpha = 1$.

(5) The wave surface varies gradually which is similar to stating that the pressure distribution is hydrostatic.

(6) Flow is subcritical so the kinematic wave is important.

2.2 Equation of Continuity:

A short reach of channel of length Δx with the flow taking place from section 1-1 to section 2-2, shown in figure 2.2.1. Assuming x to represent the horizontal distance, ρ the density of water, g, the acceleration due to gravity, z the channel bottom elevation, y, the water depth, A the cross-sectional area; and v the average velocity. If Q be the volume flow rate entering the channel through section 1-1;

$Q + (\frac{\partial Q}{\partial x}) \Delta x$ will be the volume outflow rate; the mass f water entering the channel reach during Δt time will be $\rho . Q . \Delta t$.

The mass of water leaving the channel during the same time would be

$$\rho (Q + \frac{\partial Q}{\partial x} \Delta x) \Delta t.$$

Assuming Q to increase with x i.e. $\partial Q/\partial x$ is +ve there is a net mass outflow from the reach. Applying

law of mass conservation during time Δt, it can be easily concluded that mass inside the reach will be reduced and consequently the water surface should fall. The mass of water inside the reach of length Δx is

$$\rho A \Delta x.$$

The rate of decrease of this mass is

$$- \frac{\partial}{\partial t} (\rho A \Delta x) = - \rho \Delta x \frac{\partial A}{\partial t}$$

The reduction in mass during time interval t is

$$- \rho \Delta x \left(\frac{\partial A}{\partial t} \right) \Delta t$$

For the conservation of mass,

the net mass outflow = Reduction in mass inside the

$$\rho \left(Q + \frac{\partial Q}{\partial x} \Delta x\right) \Delta t - (\rho Q \Delta t) = - \rho \frac{\partial A}{\partial t} \Delta t \Delta x$$

which gives on simplification

$$\frac{\partial Q}{\partial x} = - \frac{\partial A}{\partial t} \qquad 1$$

Since Q = AV

$$V \frac{\partial A}{\partial x} + A \frac{\partial V}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (1)$$

The momentum equation can be easily derived from the Newton's second law of motion which states that the time rate of change of momentum is equal to the external force applied, both being considered in the some direction.

Referring again to Fig. 2.1, the applied forces on the element are pressure force, gravity and frictional forces. If water depth is y at section 1-1, water depth at 2-2 will be $y + \frac{\partial y}{\partial x} \Delta x$ and similarly area at 1 -1 is A, area at 2-2 is $A + \frac{\partial A}{\partial x} \Delta x$ assuming higher order terms f Taylor's series vanish or are negligibly small.

Pressure at section 1 - 1 acting to right $= \rho g \,.\, y$

and the same at 2-2 acting to left,

$$= \rho g \left(y + \frac{\partial y}{\partial x} \Delta x \right)$$

net pressure force $= \rho gy - \rho g(y + \frac{\partial y}{\partial x} \Delta x)$

$$= - \rho g \frac{\partial y}{\partial x} \Delta x.$$

Unbalanced pressure force is

$$= - \rho g A \frac{\partial y}{\partial x} \Delta x$$

This will act towards the negative x-axis.

The component of the gravity force which is in the direction of motion is

$$= \rho g A \Delta x \quad \frac{\Delta z}{\Delta x}$$

but $\frac{\Delta z}{\Delta x}$ = channel bottom slope So.

Therefore, the gravity force in the direction of motion is $\rho g A S_0 \Delta x$. The shear force opposes the motion and is directed toward the left.

Shear force can be expressed in terms of the head loss as follows,

$$\tau P \Delta x = \gamma h_L A$$

Where P is wetted perimeter and h_L is the head loss over distance Δx.

$$h_L = S_f \Delta x.$$

So, $\tau P \Delta x = \gamma S_f \Delta x A.$

The resultant force on the element of volume in the direction of motion is the resultant of the graistational, frictional, and pressure forces:

$$\rho g A S_o \Delta x - \rho g A S_f \Delta x - \rho g A \frac{\partial y}{\partial x} \Delta x$$

The total rate of change of momentum is the sum of the local and convective momentum changes.

The acceleration of the fluid element is sum of local and convertive acceleration

$$\frac{dV}{dt} = \frac{\partial V}{\partial t} + \frac{V \partial V}{\partial x}$$

Therefore the product of mass $\rho A \Delta x$ and acceleration will give the rate of change of momentum i.e.

$$\rho A \Delta x \left(\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} \right) = \rho g A S_o \Delta x$$

$$- \rho g A S_f \Delta x - \rho g A \frac{\partial y}{\partial x} \Delta x$$

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} = g S_o - g \frac{\partial y}{\partial x} - g S_f \qquad (2)$$

Equations (1) and (2) are the equations of unsteady flow. They serve as the basis of constructing the mathematical models for the investigation of numerous problems in unsteady flow.

2.3 Solution of Governing Equations:

2.3.1 Complete Solutions:

There are 3 approaches for solving the governing differential equations (1) analytical solution (2) approximate numerical solution (3) complete numerical solution.

It is only recently with the advent of high speed computers it became possible to get the complete numerical solution in a comparatively less time and with greater accuracy.

Numerical methods for the solution of differential equations can be established by replacing the derivatives by finite differences. The differential equations are then represented by corresponding algebraic equations. In spite of the fact that numerical methods donot require any drastic simplifications of the equations of unsteady flow, the procedure for representing the partial differential equations by algebraic

equations is not straight forward. Many factors involving convergence, stability, accuracy and efficiency come into picture. Although the procedure involved may be reduced to simple mathematical operations, the number of those operations may be extremely large, and this is true for application to most flood routing problems.

Methods for the numerical solution of the complete equations of unsteady flow based on finite difference schemes are broadly classified as direct methods and characteristic methods. Direct methods are those in which the finite difference representation is based directly on the primary equations. In the characteristic methods, the equations are first transformed in to the characteristic from and the latter form is used in finite difference representation. A fixed mesh of points on the time-distance plane is employed to identify the points at which solutions are obtained by the direct methods. In the characteristic methods, solutions may be obtained at the intersection of characteristic curves on the time-distance plane or at fixed points of rectangular mesh by interpolation between the intersection of characteristics.

The finite difference schemes used in the direct and characteristic methods are further divided into two - i.e. explicit and implicit methods. In the explicit methods, the finite difference equations from which the unknowns can be evaluated explicitly a few at a time. The unknowns occur implicitly in the finite difference equations of implicit methods, which are usually non-linear algebraic equations.

Methods of Solution

- Direct
  - Explicit
  - Implicit
- Characteristic
  - Characteristic nodes
    - Explicit
    - Implicit
  - Fixed mesh
    - Explicit
    - Implicit

Out of all these methods, only three are more commonly used.

1. the implicit characteristic method using characteristic network.

2. the explicit direct method.

3. the implicit direct method.

The other methods have been found to be either unstable for general flood routing problems or they were basically similar to one of the three methods discussed above.

2.3.2 Implicit Method

In the implicit method, x derivatives are replaced in terms of finite differences evaluated at time to + Δt and thus the unknowns occur implicitly in the resulting algebraic equations, solution of these algebraic equation is more complex as compared to the explicit method.

There are many schemes which have been reported in the literature. Of these the following have been used for studying the open channel transients:

Priessman's scheme [25], Amein's scheme [4], Vasilev's scheme [26] etc.

While writing the expression for partial derivative, a weighting factor α is introduced in the

Priessmann's and in Amein's schemes. The presence of α introduces artificial damping in addition to the damping due to friction and other losses. To make the scheme stable, value of α must lie between 0.5 to 1. Vasiliev's scheme has two steps and thus is more complicated and requires more computer time. Keeping in view the simplicity of Amein's scheme, it was preferred to other ones.

McZayaney and Song have used characteristic method. But in this study implicit scheme was preferred. The reasons are following:

(1) There is no restriction on Δt in this method. Since hydrographs with different rate of rise were attempted it was necessary to have some method which does not depend much on the nature of input data.

(2) Economy: Because the size of Δt is not stricted by any stability criterion, computer space requirement is less.

2.3.3 Details of Implicit Scheme

On the (x,t) plane, the net points are determined by the intersection of straight lines parallel to the x and t axes. The lines drawn parallel to the t - axis represent locations along the channel while those drawn parallel to the x-axis represent time. The spacing of locations has been kept constant for the present analysis. The t axis is the upstream channel boundary. The downstream boundary condition is steady flow.

The net work is drawn in Fig. 2.1. With reference to the Fig. 2.1, assumed that all the variables are known at all points of network on the row t^j i.e. time step t^j and that it is desired to advance the computation to time step

$$t^{j+1} = t^j + \Delta t$$

Here, the partial derivative at central point. M of the network is defined as follows:

$$\alpha(M) = \frac{1}{4}(\alpha_i^j + \alpha_{i+1}^j + \alpha_i^{j+1} + \alpha_{i+1}^{j+1})$$

$$\frac{\partial\alpha}{\partial x}(M) = \frac{1}{2\,\Delta x}\left[(\alpha_{i+1}^j + \alpha_{i+1}^{j+1}) - (\alpha_i^j + \alpha_i^{j+1})\right],$$

$$\frac{\partial\alpha}{\partial t}(M) = \frac{1}{2\,\Delta t}\left[(\alpha_i^{j+1} + \alpha_{i+1}^{j+1}) - (\alpha_i^j + \alpha_{i+1}^j)\right]$$

where α may represent V, y, A, S_f or B. In this conversion, the continuity equation now transforms to

$$\frac{1}{2\,\Delta t}\left[(y_{i+1}^{j+1} + y_i^{j+1}) - (y_{i+1}^j + y_i^j)\right] + \frac{1}{2\Delta x} V_{i+1/2}^{j+1/2}\left[(y_{i+1}^{j+1} + y_{i+1}^j)\right.$$

$$\left. - (y_i^{j+1} + y_i^j)\right] + \frac{1}{2\Delta x}(A/B)_{i+1/2}^{j+1/2}\left[V_{i+1}^j + V_{i+1}^{j+1})\right.$$

$$\left. - (V_i^j + V_i^{j+1})\right] = 0$$

and momentum equation is,

$$\frac{g}{2\,\Delta x}\left[(y_{i+1}^j + y_{i+1}^{j+1}) - (y_i^j + y_i^{j+1})\right] + \frac{1}{2\Delta t}\left[(V_{i+1}^{j+1} + V_i^{j+1})\right.$$

$$\left. - (V_i^j + V_{i+1}^j)\right] + \frac{1}{2(\Delta x)} V_{i+1/2}^{j+1/2}\left[(V_{i+1}^{j+1} + V_{i+1}^j)\right.$$

$$\left. - (V_i^j + V_i^{j+1})\right] + \frac{g}{4}(S_{fi}^j + S_{f_{i+1}}^j + S_{fi}^{j+1} + S_{fi+1}^{j+1})$$

$$- \frac{g}{\Delta x}(z_i^j - z_{i+1}^j) = 0$$

where

$$V_{i+1/2}^{j+1/2} = \frac{1}{4}\left(V_i^j + V_{i+1}^j + V_i^{j+1} + V_{i+1}^{j+1}\right)$$

$$(V/A)_{i+1/2}^{j+1/2} = \frac{1}{4}\left[(V_i^j/A_i^j) + (V_{i+1}^j/A_{i+1}^j) + (V_i^{j+1}/A_i^{j+1}) + (V_{i+1}^{j+1}/A_{i+1}^{j+1})\right]$$

$$(q/B)_{i+1/2}^{j+1/2} = \frac{q}{4}\left[\left(\frac{1}{B_i}\right)^j + \frac{1}{B_{i+1}}^j + \frac{1}{B_i}^{j+1} + \frac{1}{B_{i+1}}^{j+1}\right]$$

$$(A/B)_{i+1/2}^{j+1/2} = \frac{1}{4}\left[(A_i^j/B_i^j) + (A_{i+1}^j/B_{i+1}^j) + (A_i^{i+1}/B_i^{j+1}) + (A_{i+1}^{j+1}/B_{i+1}^{j+1})\right]$$

For friction slope at any point, Manning's formula is used in the above formulae all the variables with j superscript are unknown while those with $j+1$ are unknown.

These equations constitute a system of two nonlinear algebraic equations in four unknowns. By themselves they are not sufficient to evaluate the unknowns at points i, j+1 and i+1, j+1.

Considering n points on any row, there will be n-1 rectangular grids and n-1 interior points. The combination of all the rectangular grids provides 2(n-1) equations for 2n unknowns. The two more equations are provided by the u/s and d/s boundary.

Now, the channel reach is simulated by N-ordinate lines on the x-t planes, a system of 2N nonlinear algebraic equations for the solutions of 2N unknowns is obtained. They are as follows:

$$F_o\ (y_1,\ V_1)\ = 0$$

$$F_1\ (y_1,V_1,\ y_2V_2\ ) = 0$$

$$G_1\ (y_1,V_1,y_2\ V_2)\ = 0$$

$$-\ -\ -\ -\ -\ -$$

$$F_{n-1}\ (y_{n-1},\ V_{n-1},\ \ y_n,\ V_n) =\ 0$$

$$G_{n-1}\ (y_{n-1},\ V_{n-1},\ y_n\ V_n)\ \ =\ 0$$

$$F_N(y_N,\ V_n)\ =\ 0$$

General Equations

In the system of equations, the values of the variables at time step t^j are known and may be treated as constants. The unknowns are denoted by superscript $(j+1)$.

The upstream boundary condition is

$$y_1 = \lambda \qquad\qquad y_1 - \lambda = 0$$

$$d_{y1} = 0$$

The downstream boundary condition is

$$F_N(y_N, V_N) = Y_N - f_N(Q_N) = 0$$

For intermediate stations, equation of continuity takes the form,

$$F_i\left[y_o, V_i, y_{i+1}, V_{i+1}\right] = (y_{i+1} + y_i) + a$$

$$+ \frac{1}{4}\frac{\Delta t}{\Delta x}\Big[(y_{i+1} - y_i)(V_i + V_{i+1} + b) + C(V_i + V_{i+1})$$

$$+ d\Big] + \frac{1}{4}\frac{\Delta t}{\Delta x}\left[(A_i/B_i) + (A_{i+1}/B_{i+1})\right]$$

$$\left[(V_i + V_i + e)\right] + \frac{1}{4}\frac{\Delta t}{\Delta x}\left[h V_{i+1} + m V_i + p\right] = 0$$

and the equation of motion is

$$G_i\left[y_i,\ V_i,\ y_{i+1},\ V_{i+1}\right]$$

$$=(y_{i+1} - y_i + a') - \frac{\Delta x}{g\,\Delta T}(V_i + V_{i+1} + b')$$

$$+ \frac{1}{4g}(V_{i+1}^2 + c'\,V_{i+1} + d'\,V_i - V_i^2 + e')$$

$$+ \frac{\Delta x}{2}(S_{fi} + S_{f\,i+1} + h') - 2.\Delta x\,(z_i - z_{i+1})$$

$$= 0$$

Here all non constant terms indicate the time step j + 1.

where a, b, c, d, e, h, p, w, a', b', c', d', e', h' are given interms of the variables at time step t^j.

$$a = -(y_{i+1}^j + y_i^j) \qquad a' = y_{i+1}^j - y_i^j$$

$$b = V_i^j + V_{i+1}^j \qquad b' = -(V_{i+1}^j + V_i^j)$$

$$c = y_{i+1}^j - y_i^j \qquad c' = 2V_{i+1}^j$$

$$d = (V^j_{i+1}(y^j_{i+1} - y^j_i) + V^j_i \; (y^j_{i+1} - y^j_i) \qquad d' = -2V^j_i$$

$$e = V^j_{i+1} - V^j_i \qquad e' = (V^j_{i+1})^2 - (V^j_i)^2$$

$$h = A^j_i / B^j_i + A^j_{i+1} / B^j_{i+1} \qquad h' = S^j_{fi} + S^j_{f_{i+1}} + 2(z^j_i - z^j_{i+1})$$

$$m = -\left(\frac{A^j_i}{B^j_i} + \frac{A^j_{i+1}}{B^j_{i+1}} \right)$$

Now the equation comprise of 2n nonlinear algebraic equation in 2n unknowns. They are

$$y_1 - \lambda = 0$$

$$F_1(y_1, V_1, y_2, V_2) = 0 \qquad G_1 (y_1, V_1, y_2, V_2) = 0$$

$$F_2(y_2, V_2, Y_3, V_3) = 0 \qquad G_2 (y_2, V_2, y_3, V_3) = 0$$

$$F_{N-1} (y_{N-1}, V_{N-1}, Y_N, V_N) = 0 \qquad G_{N-1}(y_{N-1}, V_{N-1}, Y_N, V_N)$$

$$F_N (y_N, V_N) = 0$$

In fact it is a very useful information that although the equations involve 2n unknowns, yet each equation contains a maximum of four unknowns.

Generalised Newton's Iteration Method

The application of generalised Newton's iteration method to the system of equations is made by assigning trial values to the unknowns.

$$\left.\begin{aligned}
\phi_1 (\sigma_1, \sigma_2, \sigma_3, \ldots\ldots \sigma_{2N}) &= 0 \\
\phi_2 (\sigma_1, \sigma_2, \sigma_3, \ldots\ldots \sigma_{2N}) &= 0 \\
- \quad - \quad - \quad - \quad - \quad - \quad - \quad - & \\
- \quad - \quad - \quad - \quad - \quad - \quad - \quad - & \\
\phi_{2N} (\sigma_1, \sigma_2, \sigma_3, \ldots\ldots \sigma_{2N}) &= 0
\end{aligned}\right] \quad A$$

Solutions are obtained by adjusting the trial values until residual vanishes or is reduced to a tolerable quantity. Let us indicate by suffix k the kth iterative value of σ_1, σ_2, σ_3 etc. It is desired to approximate the values of the variables through the (K+1)th cycle. When the values of the

variables are substituted into equation - - - A, the right side becomes the residuals. Let the residual be represented by R_i^k.

then

$$\phi_1 (\sigma_1^k, \sigma_2^k, \sigma_3^k, \ldots\ldots\ldots\ldots, \sigma_{2N}^k) = R_1^k$$

$$\phi_2 (\sigma_1^k, \sigma_2^k, \sigma_3^k, \ldots\ldots\ldots\ldots, \sigma_{2N}^k) = R_2^k$$

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$$\phi_{2N} (\sigma_1^k, \sigma_2^k, \sigma_3^k, \ldots\ldots\ldots\ldots \sigma_{2N}^k) = R_{2N}^k$$

According to the generalised Newton's iteration method, the values of the variables are related by the following equations

$$\frac{\partial \phi_1}{\partial \sigma_1} d\sigma_1 + \frac{\partial \phi_1}{\partial \sigma_2} d\sigma_2 + \ldots\ldots\ldots + \frac{\partial \phi_1}{\partial \sigma_{2N}} d\sigma_{2N} = R_1^k$$

$$\frac{\partial \phi_2}{\partial \sigma_1} d\sigma_1 + \frac{\partial \phi_2}{\partial \sigma_2} d\sigma_2 + \ldots\ldots\ldots + \frac{\partial \phi_2}{\partial \sigma_{2N}} d\sigma_{2N} = R_2^k$$

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$$\frac{\phi_{2n}}{\partial \sigma_1} d\sigma_1 + \frac{\partial \phi_{2N}}{\partial \sigma_2} d\sigma_2 + \ldots\ldots + \frac{\partial \phi_{2N}}{\partial \sigma_{2N}} d\sigma_{2N} = R_{2N}^k$$

where all the partial deviatives are evaluated at the kth cycle,

$$d\sigma_1 = \sigma_1^{K+1} - \sigma_1^{k}$$

$$d\sigma_2 = \sigma_2^{k+1} - \sigma_2^{k}$$

$$- - - - - - - - - - -$$

$$- - - - - - - - - - -$$

$$d\sigma_{2N} = \sigma_{2N}^{k+1} - \sigma_{2N}^{k}$$

Now the above equation can be solved for $d\sigma_1$, $d\sigma_2$, ... $d\sigma_{2N}$ and values of σ_1^{k+1}, σ_2^{k+1}, ... σ_{2N}^{k+1} can be easily found out. This procedure is calculated till the residual is negligible.

Solution of the finite difference Equations

The same procedure can now be illustrated with the flood routing problem.

The e-uations are:

$$y_1 - \lambda = 0$$

$$F_1 \ (y_1, V_1^k, \ y_2^k, V_2^k) = R_{1,1}^k$$

$$G_1 \ (y_1 V_1^k, y_2^k, V_2^k) = R_{2,1}^k$$

$$- \quad - \quad - \quad - \quad - \quad -$$

$$- \quad - \quad - \quad - \quad - \quad -$$

$$F_i \ (y_i^k, \ V_i^k, \ y_i^k, \ V_i^k) = R_{1,i}^k$$

$$G_i \ (y_i^k, \ V_i^k, \ y_i^k, \ V_i^k) = R_{2,i}^k$$

$$- \quad - \quad - \quad - \quad - \quad -$$

$$F_{N-1} \ (y_{N-1}^k, \ V_{N-1}^k, \ y_N^k \ V_N^k) = R_{1,N-1}^k$$

$$G_{N-1} \ (y_{N-1}^k, \ V_{N-1}^k, \ y_N^k, \ V_N^k) = R_{2,\ N-1}^k$$

$$F_N \ (y_N^k, \ V_N^k) = R_{1,N}^k$$

The residuals and the partial derivatives are related according to the generalised iteration method by

$$dy_1 = 0$$

$$\frac{\partial F_1}{\partial y_1} = dy_1 + \frac{\partial F_1}{\partial V_1} dV_1 + \frac{\partial F_1}{\partial y_2} + \frac{\partial F_1}{\partial V_2} dV_2 = R^k_{1,1}$$

$$\frac{\partial G_1}{\partial y_1} dy_1 + \frac{\partial G_1}{\partial V_1} dV_1 + \frac{\partial G_1}{\partial y_2} dy_2 + \frac{\partial G_1}{\partial V_2} dV_2 = R^k_{2,1}$$

- - - - - -

- - - - - -

$$\frac{\partial F_{N-1}}{\partial y_{N-1}} dy_{N-1} + \frac{\partial F_{N-1}}{\partial V_{N-1}} dV_{N-1} + \frac{\partial F_{N-1}}{\partial y_N} dy_N + \frac{\partial F_{N-1}}{\partial V_N} dV_n = R^k_{1,N-1}$$

$$\frac{\partial G_{N-1}}{\partial y_{N-1}} dy_{N-1} + \frac{\partial G_{N-1}}{\partial V_{N-1}} dV_{N-1} + \frac{\partial G_{N-1}}{\partial y_N} dy_N + \frac{\partial G_{N-1}}{\partial V_N} dV_N = R^k_{2,N-1}$$

$$\frac{\partial F_N}{\partial y_N} dy_N + \frac{\partial F_N}{\partial V_N} dV_N = R^k_{1,N}$$

where

$$dy_1 = 0 , \quad dV_1 = V_1^{k+1} - V_1^k$$

$$dy_i = y_i^{k+1} - y_i^k, \quad dV_i = V_i^{k+1} - V_i^k$$

- - - - - - -

$$dy_N = y_N^{k+1} - y_N^k, \quad dV_N = V_N^{k+1} - V_N^k$$

All the partial derivatives are evaluated at the kth iteration cycle. The solution of the system of equation will provide values of y_i^{k+1}, V_i^{k+1}. The procedure will be repeated many times until the desired accuracy is achieved. The values of the variables found in the terminal iteration cycle will be taken as the values of the variables for the time step (j+1) and the computations will be advanced to next time step (j+2).

The coefficients of the matrix are obtained by diffentiating equations with respect to the independent variables and are given below.

$$\frac{\partial F_i}{\partial y_i} = 1 - \frac{1}{4} \frac{\Delta t}{\Delta x} (V_i + V_{i+1} + b) + \frac{1}{4} \frac{\Delta t}{\Delta x} (V_{i+1} - V_i + e) \left[1 - \left(\frac{A_i}{B_i^2}\right)\left(\frac{dB_i}{dy_i}\right)\right]$$

$$\frac{\partial F_i}{\partial y_{i+1}} = 1 + \frac{1}{4} \frac{\Delta t}{\Delta x} (V_i + V_{i+1} + b) + \frac{1}{4} \frac{\Delta t}{\Delta x} \left[1 - \left(\frac{A_{i+1}}{B_{i+1}^2}\right)\left(\frac{dB_{i+1}}{dy_{i+1}}\right)\right] (V_{i+1} - V_i + e)$$

$$\frac{\partial F_i}{\partial V_i} = \frac{1}{4} \frac{\Delta t}{\Delta x} \left[(y_{i+1} - y_i) + c \right] - \frac{1}{4} \frac{\Delta t}{\Delta x} \left[(A_i/B_i) + \left(\frac{A_{i+1}}{B_{i+1}} \right) \right] + \frac{m}{4} \frac{\Delta t}{\Delta x}$$

$$\frac{\partial F_i}{\partial V_{i+1}} = \frac{1}{4} \frac{t}{x} \left[(y_{i+1} - y_i) + c \right] + \frac{1}{4} \frac{\Delta t}{\Delta x} \left[(A_i/B_i) + \left(\frac{A_{i+1}}{B_{i+1}} \right) \right] + \frac{h}{4} \frac{\Delta t}{\Delta x}$$

$$\frac{\partial G_i}{\partial y_i} = -1 + \frac{2}{3} (\Delta x) \, Sf_i \left[\frac{1}{B_i} \left(\frac{dB}{dy} \right)_i - \frac{B_i}{A_i} \right]$$

$$\frac{\partial G_i}{\partial y_{i+1}} = 1 + \frac{2}{3} \Delta x \, S_{fi+1} \left[\frac{1}{B_{i+1}} \left(\frac{dB}{dy} \right)_{i+1} - \frac{B_{i+1}}{A_{i+1}} \right]$$

$$\frac{\partial G_i}{\partial V_i} = \frac{1}{g} \frac{\Delta x}{\Delta t} - \frac{1}{2g} V_i + \frac{d'}{4g} \frac{\Delta x}{V_i} S_{fi}$$

$$\frac{\partial G_i}{\partial V_{i+1}} = \frac{1}{g} \frac{\Delta x}{\Delta t} + \frac{1}{2g} \left[V_{i+1} + \frac{c'}{2} \right] + \frac{\Delta x}{V_{i+1}} S_{f_{i+1}}$$

This numerical procedure was applied for routing the flood through the channel.

2.3.4 Verification of the Programme

In the book Hydraulic Transients by Streeter and Wyle, in the Chapter open channel flow a problem of flood routing has been solved. The same problem was taken for checking the correctness of the program. The results obtained by the implicit method were compared with the solution obtained in the book. It was found that the depth compared very well in the two solutions. Velocity in the implicit scheme was higher than the characteristic method. But the difference was limited to the second decimal places. This validates the computer program.

CHAPTER III

LITERATURE REVIEW

The subsidence of wave in the channel with initial steady condition is the phenomenon of prime interest for any practicising engineer. The subsidence may be controlled by the resistance and acceleration terms in the dynamic equations of motion or by a much more simple mechanism of pondage in lakes through which the flood passes. But before going to the fundamentals and detailed analysis, it would be useful to have an idea of how other analysers have explained this phenomenon.

The simplest wave form as given by Seddon [21] or independently by Kleitz, is the monoclinal rising wave in a uniform channel. Scuh a wave consists of an initial steady flow, a period of uniformly increasing flow, and a continuing steady flow at the higher rate. Superimposing on this wave system a velocity v equal and opposite to the wave celerity, c causes the wave to become stationary and a steady flow known as overrun

is = $(c - v_1)A$ or $q - cA$ = constant

$$\text{or} \qquad \frac{dq}{dA} = c = \frac{1}{B}\frac{dq}{dy}$$

This is called Seddon law after the man who demonstrated its validity on the Mississippi River [21]. This Seddon's law is based on the assumption that q is a function of y alone. Secondly, it assumes that the wave form does not subside or diminish in shape which is contrary to the experience. To overcome these defects, concept of kinematic wave was introduced by Lighthill and Whitham [19]. These investigators defined a kinematic wave as one in which Q is a function of y alone; this imples $S_f = S_o$ and that the other slope terms are negligible. It can be easily shown that there can be a true wave motion when Q is a function of y alone, by considering the equation continuity i.e.

$$\frac{\partial Q}{\partial x} + B\frac{\partial y}{\partial t} = 0$$

$$\text{or} \qquad \frac{\partial Q}{\partial y}\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = 0$$

$$\text{or} \qquad \frac{dy}{dt} = \frac{dx}{dt} \cdot \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0$$

$$\text{where} \qquad \frac{dx}{dt} = C = \frac{1}{B}\frac{dQ}{dy}$$

Therefore, for an observer moving with a velocity c, y will appear to be constant and so Q will also appear to be constant. This leads to a very important conclusion that the Kinematic waves move with the same velocity as given by the monoclinal wave. At this stage dynamic wave can be introduced. This wave is obtained from the dynamic momentum equation and continuity equation. The kinematic waves lead to only one wave velocity while far the dynamic waves there are two possible wave speeds physically this means that a dynamic disturbance i.e. one in which all slope terms are important will propagate in both upstream and downstream directions. A kinematic disturbance will propagate in the downstream direction.

Actually there is possibility that both the kinds of wave motion are present in any natural flood wave. S ince the bed slope term dominates the other slope terms, even if the latter ones are negligible, the main bulk of the flood moves substantially as a kinematic wave, although its character is modified by the other slope terms. Despite these arguments, the speed of the main flood wave may be expected to approximate

to that of the kinematic wave and the same has been concluded by Lighthill and Whitham.

Henderson [14] in his papers white discussing the flood wave characteristics has shown that the leading dynamic wave acts as a fore runner of the main wave, and if $F_O > 2$ this fore-runner will bring about appreciable rise of the water level before the arrival of the main wave. But in most of the natural flood wave problems, F_O is seldom greater than 2 and so fore runner does not play any singnificant role. In fact the discussion of the fore-runner is of limited practical limited utility while dealing with the natural flow problems. Henderson has shown by qualitative analysis that there will be appreciable subsidence of the flood wave. The space rate of subsidence is itself a small fraction of the bed slope. In the most of the flood routing problems, to avoid time consuming calculations, a much simplified approach was adopted by the practicising engineers. The flood wave movement can be simulated by an electric analogy in which a pulse is fed into the system. This pulse can be attenuated either by a capacitance or by a

resistance. When effect of capacitance is dominant or hydraulically, when effect of storage is important it is called storage routing. Thomas [28] used graphical method of integration of the governing equations namely continuity and momentum equation in his analysis. He used the simplified form of equation of motion by neglecting second order terms. This method requires the contour map of the area and is also crippled with tedious time consuming graphical approach.

The best known approximate methods for the integration of the equations of unsteady flow are the so called storage routing [20] methods. In a typical method the equation of continuity without lateral flow is represented in finite difference form as

$$\frac{\Delta Q}{\Delta x} = \frac{\Delta A}{\Delta t} \quad \text{or} \quad \Delta Q = \frac{\Delta A}{\Delta t} \cdot \Delta x$$

$\Delta A \cdot \Delta x$ is nothing but the change in the volume or the change in storage i.e.

$$\Delta S = \Delta A \cdot \Delta x \quad \text{where } S = \text{storage}$$

$$\Delta Q = \frac{\Delta S}{\Delta t}$$

$$O - I = \frac{\Delta S}{\Delta t}$$

This equation is the equation of continuity. Another widely used storage routing method is the Muskingum method [20]. This is called Muskingum method because it was applied in the river Muskingum at Ohio. In this method the storage-discharge relationship is given by an empirical equation the coefficients of which are obtained from historical records. This method is described in any of the text books [20, [illegible]]. Brakensiek and Comer (1966) gave an approximate approach in which the equation of continuity is retained and the momentum equation is simplified to $(\partial y/\partial x) = S_o - S_f$ by neglecting the accelerating terms.

Approximate methods whether numerical or analytical provide solutions which are useful and accurate but the serious disadvantage is as regards the uncertainty in accuracy of solution. With the use of high speed computers, attempts were made to solve the complete equations numerically. The one such attempt was made by M. Amein in 1966 [3], Freed (1973), Baltzer etc. These methods were computer based and so the degree of accuracy can be taken to any desired degree of accuracy. Amein [4] in report has shown how successfully

the implicit scheme can be used for an irregular channel. Out of all the methods namely explicit, characteristics and implicit, the last one has been found to be most stable.

The parametric study of flood wave propagation in a part full pipe was done by Peter Ackers and A.J.M. Harrison [1] in 1964. The work was published in the Proceedings of the Institution of Civil Engineers 1964, Vol. 28. Their study was for a pipe flow. They studied only the subsidence of peak depth. In 1969, Mozayaney and Song also presented a parametric study on the flood propagation. They used a sinusoidal hydrograph. The wave amplitude and duration etc. were very small so that they are most suited for a small flume.

In a very recent paper, Sridharan and Mohan Kumar [26] studied the effect of various parameters in the case of open channels. They have nondimensionalised the equations of continuity and equation of momentum. This study is a good approach towards the parameteric studies. But they covered a lot of dependent variables and thus complicating the study.

CHAPTER IV

FORMULATION OF THE PROBLEM

4.1 General

It is obvious from the theory that it is very difficult to solve the governing equations analytically for various practical problems. The numerical techniques are very complicated and for accuracy high speed computer is required. Keeping in view all these things it was considered desirable to carry out a systematic dimensional analysis and determine the variation of the important dimensionless parameters relating to flood propagation as the pertinent independent parameters are varied systematically.

4.2 Dimensional Analysis

4.2.1 General

One of the important dependent variables of interest in flood routing through a river is the subsidence or decay of the wave amplitude with distance. This subsidence is defined as the depth at any section minus initial steady depth, non-dimensionalised by the net increase in stage at the initial station. In other words this term gives the wave amplitude at any section normalised by the

initial wave amplitude at x = 0. Mathematically,

$$Y_* = \text{Relative wave amplitude}$$
$$= \frac{(Y_p)_{x=i} - Y_o}{(Y_p)_o - Y_o}$$

The other important variable is the time of travel of the peak. The time lapse between the occurrence of peak stage at the initial station and at any stations(downstream) is equal to the time of travel of the peak. The time of peak depth, expressed as T_p is non-dimensionalised by $x/(V_o+C_o)$ which is the theoretical time taken by the wave to reach at particular location. This non-dimensionalised time presents an idea of how the wave is moving in time space plane. Too large value of it means that small amplitude waves are coming first and peak of the hydrograph has not been reached. Similarly if it becomes small, it means the wave is moving with the peak depth.

4.2.2 Dimensional Analysis

The independent variables are the initial uniform flow depth Y_o, distance of the section under consideration x, Manning's n, initial Froude No F_o, time of peak of the hydrograph t_p, time to centre of gravity of the hydrograph t_g, peak depth of the hydrograph y_w, rate of rise of

the rising limb described as y_w/t_p, nature of the hydrograph.

Mathematically,

$$Y_* = f_1(x, y_o, n, y_p, F_o, t_p, t_g, \frac{y_w}{t_p}, \text{shape of the hydrograph})$$

$$\frac{T_p}{x/(V_o+C_o)} = f_2(x, y_o, n, y_p, F_o, t_p, t_g, \frac{y_w}{t_p}, \text{shape of the hydrograph})$$

Dimensionless grouping of the above variables leads to the following two equations

$$Y_* = f_1(\frac{x}{y_o}, n, F_o, \frac{t_p}{t_g}, \frac{y_p}{y_o}, \frac{y_p-y_o}{t_p(V_o+C_o)}, \text{shape of the hydrograph})$$

$$\frac{T_p}{\frac{x}{(V_o+C_o)}} = f_2(\frac{x}{y_o}, n, F_o, \frac{t_p}{t_g}, \frac{y_p}{y_o}, \frac{y_p-y_o}{t_p(V_o+C_o)}, \text{shape of the hydrograph})$$

where F_o = Froude no. at initial steady flow

$$= \frac{q_o}{Y_o\sqrt{g\,y_o}}$$; g is the acceleration due to gravity $= 9.81$ m/sec.^{2}

$$V_o = \frac{q_o}{y_o}$$; q_o is initial steady discharge

$$C_o = \sqrt{g\,y_o}$$

$\frac{y_p-y_o}{t_p(V_o+C_o)}$ = dimensionless rate of rise of the hydrograph.

4.3 Parametric Study

The important factors which control the movement of flood waves in the regular channel can be classified into 3 groups.

(i) Channel Geometry

(ii) Flow Condition

(iii) Initial Conditions

The variables in each are as follows:-

1. Channel Geometry
   (a) Cross-sectional shape
   (b) Channel bed slope
   (c) Roughness of the channel expressed as Manning's coefficient n
2. Flow Condition
   (a) Upstream boundary condition- given by the inflow hydrograph
   (b) Base flow
   (c) Peak depth of flow
   (d) Time to reach peak of the hydrograph
   (e) Time to the centre of gravity of the hydrograph.
3. Initial Condition
   (a) Steady uniform flow
   (b) Steady nonuniform flow.

4.4 Range of Variables

4.4.1 Froude Number F_o

In most of the natural rivers the flow is in subcritical state and leaving the initial reach at **its** mouth , the river flows in tranquil state.

In their recent paper of ASCE [26] , the authors have concluded that the Froude number does not have significant effect on the subsidence in the case of wide channels. Keeping in view the large number of computer experiments it was decided to use the most suitable single value for Froude Number. This value has been taken as equal to 0.3.

4.4.2 Cross-section area

The simplest one is rectangular shape. So rectangular section was adopted.

4.4.3 Manning's n

Manning's n was varied from 0.02 to 0.03. It was not possible to go beyond n =0.03 for $y_w/y_o > 3$ as there was local supercritical flow.

4.4.4 Wave amplitudes

y_p/y_o: In most of the flood problems the maximum depth of flow comes out to be nearly 3 times the steady

depth. So y_p/y_o was varied from 1.5 to 3.

4.4.5 $\frac{t_p}{t_g}$

The worst flood has a value of 1/3 and the other one was 2/3.

4.4.6 Inflow hydrograph

Inflow hydrograph used are of three types:

(1) Sine curve

(2) +vely skew hydrograph of type III

(3) Triangular hydrograph adjusted in (2) above.

Sine curve was not used in the further analysis as it does not resemble the real flood problem. The suitability of +vely skew hydrograph bound in the direction of zero has been discussed with details in Chapter V.

The equation of the hydrograph used was

$$\frac{y}{y_o} = 1 + \frac{y_w}{y_o} \left[e^{-\frac{(t-t_p)}{(t_g-t_p)}} \right] \left[\frac{t}{t_p}\right]^{(t_p/(t_g-t_p))}$$

where all terms have been defined earlier.

CHAPTER V

ANALYSIS OF RESULTS

5.1 General

Based upon the dimensional analysis all the variables were non-dimensionalised. Various graphs were plotted between relative wave amplitude and distance with one more variable keeping all others constant. This sort of analysis will give an insight into the complicated phenomenon of wave propagation.

But the first question arises as to what the shape of the inflow hydrograph should be. Mozayaney and Song in their paper [26] have used sinusoidal hydrograph. But in actual field observations no hydrograph is strictly sine curve, rather they have steeply rising limb and comparatively flatly falling limb. So it was considered more justified to use skewed (to the left) hydrograph in place of symmetrical ones. That the hydrograph should be +vely skewed one can be shown from the argument which follows here. Refer to Fig. 5.1. The hydrograph at the ~~mouth~~ origin of the river is nearly symmetrical. But the falling limb

has a velocity $= V + C$ while the rising limb has $V-C$ where C is celerity $= \sqrt{gy_o}$ which of-course will die out as it travels upstream. But if temporal distribution of flood wave is considered, it can be concluded that the rising limb will remain steep but falling limb will loose its steepness very quickly. Modification of wave with distance and time is shown in figure 5.2 which also supports the statements made here.

Before proceeding to further analysis some terms are defined here.

(a) Rate of subsidence is described as

$$\text{Relative wave amplitude } Y_* = \frac{(y_{peak})_{x=i} - y_o}{(y_{peak})_{x=0} - y_o} .$$

The relative wave amplitude gives the maximum wave amplitude at any section normalised by the same at $x = 0$.

(b) Relative discharge rise

$$Q_* = \frac{Q(q_{max})_{x=i} - q_o}{(q_{max})_{x=0} - q_o}$$

(c) $$T_* = \frac{T_P}{x/(V_o + C_o)}$$

i.e. time taken by wave peak to reach any station since the starting of the hydrograph non-dimensionalised by the theoretical time taken to reach that section.

5.2 Analysis of rate of subsidence:

To analyse this following graphs were plotted

(1) Y_* VS X with different rate of rise of the hydrograph.

(2) Y_* vs X with different Y_w keeping tp/tg = constant.

(3) Y_* vs X with different tp/tg values keeping Y_w = constant

(4) Y_* Vs X with n.

5.2.1 Effect of n:

Plots of relative wave amplitude Y_* and X with Manning's n as the third parameter indicate that Mannings n has very significant effect on the subsidence.

From the graphs, it can be seen that an increase in mannings roughness coefficient result in the increase of relative wave amplitude.

M

For $n = 0.02$, $Y_* = 0.550$ for $tp/tg = 2/3$ at $x = 16$

for $n = 0.03$, $Y_* = 0.685$ for $tp/tg = 2/3$ at $x = 16$.

Therefore, the effect of Mannings n is to decrease the subsidence. When Mannings n increases the wave pass es without getting subsided. This may be explained from the momentum equation. Energy slope is directly dependent upon mannings n and so with the increase in manning's n, energy slope S_f also increases.

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{dy}{\partial x} = g\,S_o - g\,S_f$$

Now, the right hand term diminish and so do the left hand terms, i.e., the acceleration term decreases. This appears to be the possible explanation of this effect of mannings n.

5.2.2 Effect of Wave Amplitude Y_w

The variation of wave amplitude Y_* with X keeping tp/tg constant is plotted in the figure 5.3 and 5.5.

From these curves it is clear that relative wave amplitude increases with increase in Y_w. This increase is small as compared to the effect of Mannings n.

Secondly, it can be seen that the steepness of the plotted curve decreases with the increase in distance X. The wave in later reaches tend to become flatter, and the subsidence in the later reaches has been found to be more. In the initial reaches Y_* and X vary in a straight line pattern. The same conclusion has been confirmed by Sridharan [26] in his paper. In fact this straight liness is true for small length of the river. Lyer used 30 km of channel length and his conclusions also support the straight line nature.

5.2.3 Effect of tp/tg:

The plots of Y_* with X for different tp/tg values are presented Fig. 5.6 and 5.7. Y_* decreases linearly with X in the initial reaches. The line tends to become flatter with increase in distance. The rate

of change in slope decreases with distance. The effect of tp/tg is to increase the relative wave amplitude Y_*. This effect is more pronounced than the effect of wave amplitude Y_w.

Subsidence is more in the later reaches. This can be explained as follows. When the wave has flattened sufficiently in the lower reaches, the waves become short duration ones and rate of subsidence increases significantly.

5.2.4 Effect of Rate of Rise of the Hydrograph:

Rate of rise of the hydrograph is defined as yw/tp. From the computed results of rate of rise of hydrograph it was found that this term practically has no effect on the relative wave amplitude. Fig. 5.8 clearly shows this. In fact this rate of rise is taken into account jointly by yw/yo and tp/tg and so this term can be ignored.

5.2.5 Effect of shape of the Hydrograph:

Keeping tp/tg, yw/yo, n same various runs of the computer experiment were made by using triangular

hydrographs. From the computed results it was found that the shape does not play much role in subsidence phenomena. Although in case of tp/tg = 1/3, the triangular shape showed a little more subsidence than the +vely skewed hydrograph. This can be explained as follows In the case of triangular hydrographs of very steep rate of rise, the falling part gives more depth than the +ve skewed hydrographs and so the cumulative effect of the triangular hydrograph is slightly larger subsidence. This is shown in figure 5. 9. The above mentioned two conclusions are very significant. Now only n, yw/yo, tp/tg remain to be studied.

5.3 Rating Curves:

A graphical representation of stage vs discharge is called a rating curve. Such a rating curve for a typical case is plotted in Fig. 5.3(a) The rating curve clearly indicate that there are two values of stage for one discharge. On the rising side the discharge is more for the same stage as compared to the falling side. Secondly with increase in distance

looping curve tends to become thinner meaning thereby unique value of the discharge for one value of the stage. The first observation leads to the belief that the discharge is not a single valued function of stage or discharge is a function of some thing else other than the stage. In fact during the period of rise of the hydrograph discharge is more but after the peak of the hydrograph, discharge falls down owing to the reverse slope of water surface.

5.4 Analysis of Time of Occurrence of Peak Depth:

Time of occurrence of peak is defined as the time of occurrence of flood peak at a particular section in the reach minus the time of occurrence of flood peak at the initial station. This is the actual time of travel of the peak.

5.4.1 Effect of n:

From the computation of results it became clear that Manning's n plays practically no part in the time of occurrence of peak. Previously in the analysis of relative wave magnitude this very term was found to be

a dominating factor. It confirms that in the left hand terms the effect of temporal acceleration is little as compared to the spatial acceleration.

Time to peak is non dimensionalised by the theoretical time taken by the kinematic wave to reach the section and the plot indicates that $\frac{T_P}{x/(Vo+Co)}$ is independent of mannings n. Therefore it can be concluded that time to peak bears a constant ratio with the theoretical time taken by the wave to reach at the same section.

5.4.2 Effect of Y_w

Graphical representation of $\frac{T_P}{x/(Vo + Co)}$ vs X with Y_w as third variable indicates the following points. One is that in the initial reaches there is very steep fall in time to peak. This is probably due to the discontinuity of the term $T_P/(x/(Vo+Co))$ at $x = 0$. Secondly, in the later reaches the ratio begin to decrease with distance or time to peak is quickly reached at the section under consideration. Thirdly, the $T_P/(x/(Vo + Co))$ value tend to become constant in the

further reaches. Fourthly, the effect of initial wave ratio Y_W is that with the increase in Y_W, $\frac{T_P}{x/(V_o+Co)}$ increases or lag of time to peak with theoretical time increases. So it can be concluded at this stage that the effect of initial wave ratio is very significant in the analysis of time to peak. This property can be of great use in the prediction of time to peak as will be shown in the next article.

5.4.3 <u>Effect of tp/tg:</u>

If time to peak T_P is nondimensionalised by theoretical time required to travel a distance is plotted with distance x, it is found that tp/tg of the hydrograph is not very important variable.

Therefore it appears that the wave travels with a velocity which is independent of the tp/tg ratio of the hydrograph. This conclusion has been confirmed by Sridharan [20] also.

5.5 Prediction of Y_p and T_p

5.5.1 General

In any flood routing problem, the engineer is interested in finding the magnitude of the maximum stage, maximum flood and time of their occurrence. By the computer solution based on numerical methods, one gets detailed picture of the surface profile, stage at any time at any section etc. which are not of much use to a practising engineer. Further the computer programmes are very complicated and require a lot input data in an specified form. All these factors call for a direct short-cut method for the prediction of peak stage and time of its occurrence.

After the detailed study of wave propagation based on dimensional analysis it is now possible to drop one or even more of the variables as their relative effect on the dependent variables is less.

5.5.2 Principle

From the previous study in this chapter, it can be concluded that for the rate of subsidence of relative wave amplitude is not dependent upon shape of the hydrograph,

rate of rise of the hydrograph, wave amplitude Y_w. The most important variables affecting Y_* are Manning's n and t_p/t_g. A plot of $1-Y_*$ on natural scale and X on logarithmic scale indicates that after some initial points, the points lie approximately on a straight line. This plot indicates that

$$1-Y_* = Y^+ = c + m \log_{10}(X)$$

where c and m are functions of n and t_p/t_g.

i.e. $$c, m = f_{1,2}\left(n, \frac{t_p}{t_g} \right)$$

This is a very important conclusion and forms the basis of prediction of Y_* . Similarly from the analysis of time to peak stage, it is clear that it depends only upon Y_w and X. This forms the basis of prediction of time of peak stage.

5.5.3 Procedure for graphical prediction

(a) Relative wave amplitude

On a semilog paper, X is plotted on logarithmic scale as abscissae and $1-Y_* = Y+$ as ordinate on arithmatic scale.

After some of the initial points, which correspond to the straight lime portion of the Y_* vs n plot, a straight line is approximately fitted through them. The slope and intercept of the lines can be calculated by taking any two points on the straight line and solving them for m and c, where m = slope of the line, c = intercept on Y_+ axis.

$$\text{now,} \quad m, c = f_{1,2}\left(n, \frac{t_p}{t_g}\right)$$

So, m and c are plotted on a simple graph paper with n on abscissae and t_p/t_g as the third variable.

Since some error is caused due to neglecting of effect of t_p/t_g a suitable additive constant should be used for each y_w value.

(b) <u>Time of peak $T_* = \dfrac{T_p}{x/(V_o+C_o)}$</u>

A plot of T_p/t_g on ordinate and X on abscissae with y_w as the third parameter is prepared. The dotted lines on the plot indicate the interpolated variation T_p/t_g and X with other y_w values.

	Additive Constant for Y_*		
Y_w =	0.5	1	1.5
Constant to be added to Y_* =	0.015	0.02	0.035

5.5.4 Steps

Data required:

Initial steady discharge q_o , Manning's n, t_p/t_g of the inflow hydrograph. Slope of the channel s distance of the section under consideration x time to centre of gravity of the hydrograph.

(1) From Manning's equation initial steady depth should be computed:

$$\text{i.e.} \quad y_o = \left| \frac{q_o\ n}{s} \right|^{3/5}$$

(2) For the given x, $X = \dfrac{x}{y_o}$ is calculated.

(3) Using the plot of Fig. 6-2 , against t_p/t_g and n value m and c are calculated.

(4) Now,

$$Y+ = m \log_{10} X - C$$

Y+ can be calculated.

(5) $Y_*^{'} = 1 - Y^{+}$

(6) $Y_* = Y_*^{'} + \text{additive constant.}$

This additive constant has to be interpolated from the table given above.

(7) $$Y_* = \frac{(y_{peak})_{x=i} - y_o}{(y_{peak})_{x=0} - y_o}$$

from this $(y_{peak})_{x=i}$ can be calculated.

(8) Now coming to the Fig. 6.3.

From this plot for the given X and Y_w of the hydrograph, T_p/t_g can be noted down and t_g being known T_p can be calculated.

Illustration

For x = 500 Km , n = 0.02 , $Y_w = 1.0$

$$x = \frac{500 \times 10^3}{5} = 10^5 \qquad t_g = 12 \text{ hrs.}$$

for n = 0.02 and $\frac{t_p}{t_g} = \frac{2}{3}$

$m = 0.044$

$c = 1.845$

$Y+ = -1.845 + 0.44 \log_{10} 10^5 = 0.355$

$Y_*' = 1-0.355 = 0.645$

$Y_* = 0.44 + 0.02 = 0.665$

$y = 0.665 \times 5 + 5 = 8.325$

which resembles very closely with the value of 8.26 m.

For time to peak, $T_p = 12 \times 3.30 = 39.6$ hrs. 40 hrs .

From the computed results also, we get $T_p = 40$ hrs.

Limitation

(1) The magnitude of numerical damping is not known.

(2) This is strictly for skewed hydrographs but can be used safely with triangular hydrographs for t_p/t_g upto 1/3.

(3) There should not be local supercritical flow.

CHAPTER VI

6.1 Conclusions

The parametric study on the propagation of a flood wave in open channel leads to the following conclusions:-

1. There is significant subsidence in open channels.
2. Manning's roughness coefficient has significant effect on the subsidence expressed as relative wave amplitude.
3. The non-dimensional time to peak is found to be unaffected by the manning's n.
4. Shape of the hydrograph does not matter much in wave subsidence.
5. Rate of rise of the hydrograph does not affect the rate of subsidence.
6. Inflow wave amplitude Y_W has some influence on subsidence.
7. Wave peak duration t_p/t_g has pronounced effect on subsidence.
8. Non-dimensional travel time is very much affected by the wave amplitude and is not affected by t_p/t_g.

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TABLE

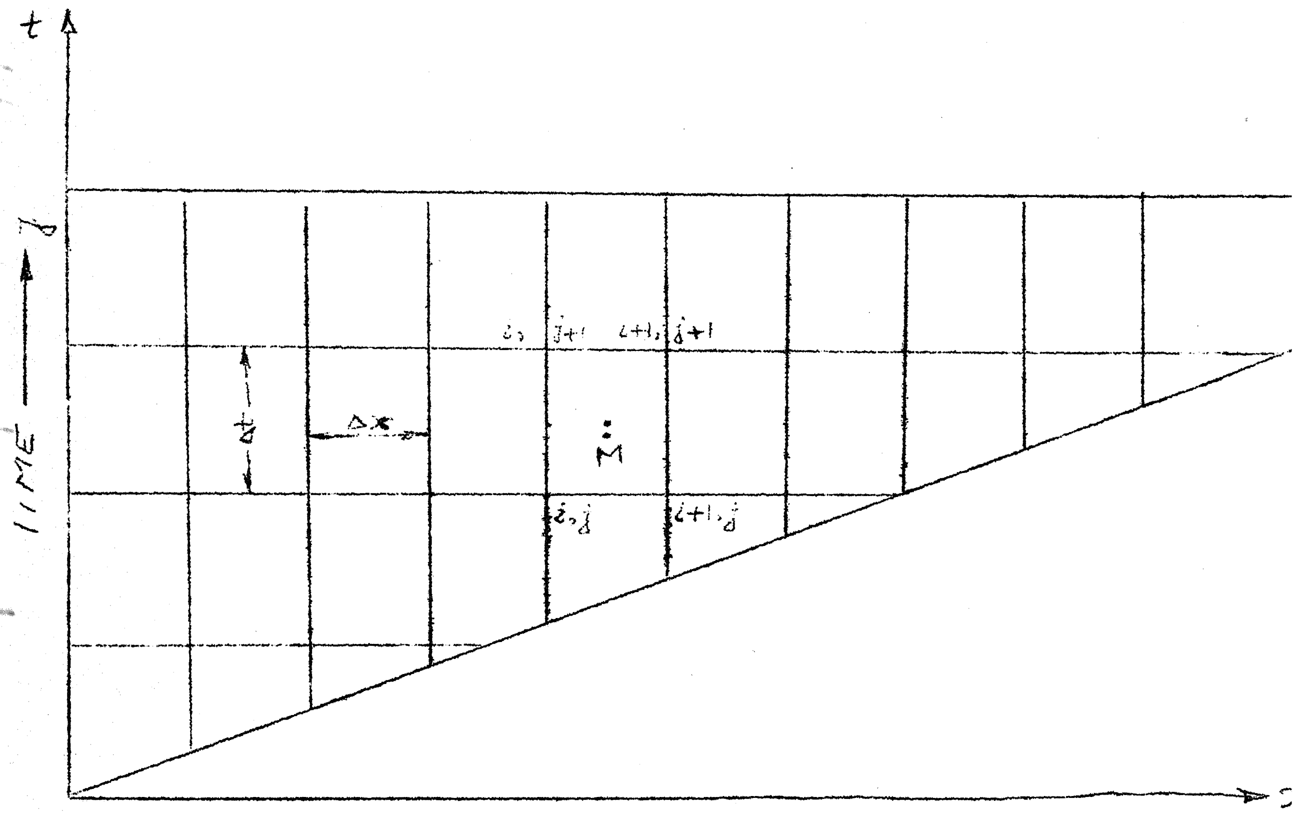
RANGE OF VARIABLES

$y_o = 5m$, $t_g = 12$ hrs

Set No.	Froude No. F_o	Wave amplitude Y_w	$\frac{t_p}{t_g}$	Manning's Roughness n	Slope S_o
1	0.3	0.5	2/3	0.02 0.025 0.03	0.21×10^{-3} 0.32×10^{-3} 0.47×10^{-3}
2	0.3	0.5	1/3	0.02 0.025 0.03	0.21×10^{-3} 0.32×10^{-3} 0.47×10^{-3}
3	0.3	1.0	2/3	0.02 0.025 0.03	0.21×10^{-3} 0.32×10^{-3} 0.47×10^{-3}
4	0.3	1.0	1/3	0.02 0.025 0.03	0.21×10^{-3} 0.32×10^{-3} 0.47×10^{-3}
5	0.3	2.0	2/3	0.02 0.025 0.03	0.21×10^{-3} 0.32×10^{-3} 0.47×10^{-3}
6	0.3	2.0	1/3	0.02 0.025 0.03	0.21×10^{-3} 0.32×10^{-3} 0.47×10^{-3}

Effect of Rate of Rise of Hydrograph

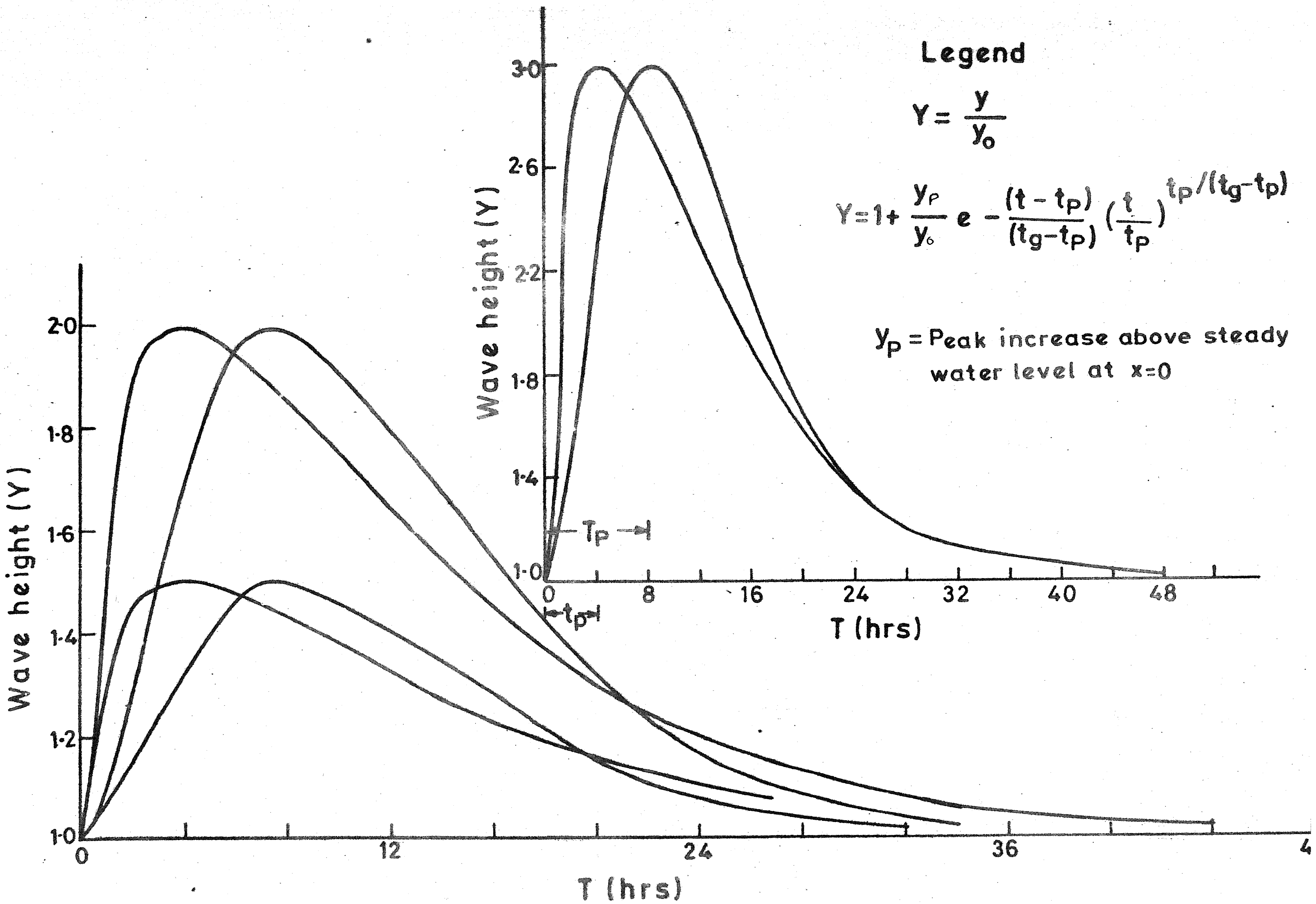
Set No.	F_o	Y_w	t_p/t_g	Rage of Rise	Manning's n
1	0.3	1	2/3	9.7×10^{-5}	0.02
2	0.3	1	2/3	7.5×10^{-5}	0.02

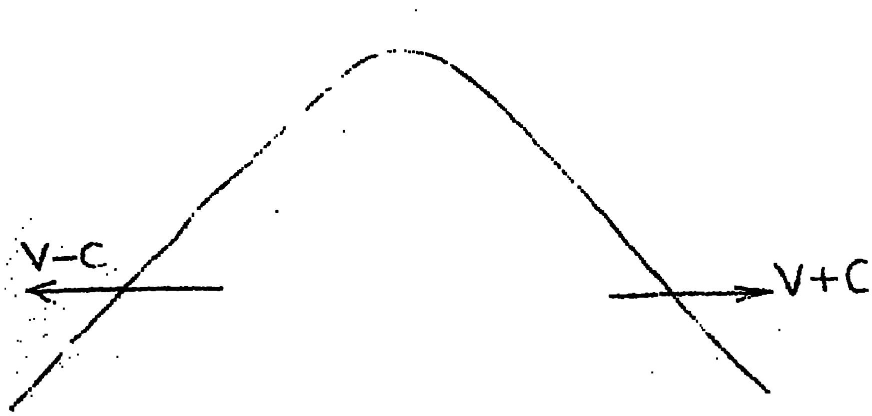


DISTANCE i →

REPRESENTATION OF NETWORK

FIG. 2.1


Legend
$Y = \frac{y}{y_o}$
$Y = 1 + \frac{y_P}{y_o} e - \frac{(t - t_P)}{(t_g - t_P)} \left(\frac{t}{t_P}\right)^{t_P/(t_g - t_P)}$
y_P = Peak increase above steady water level at x=0
Wave height (Y)
3·0
2·6
2·2
1·8
1·4
1·0
T_P
t_P
0
8
16
24
32
40
48
T (hrs)
Wave height (Y)
2·0
1·8
1·6
1·4
1·2
1·0
0
12
24
36
4
T (hrs)



SYMMETRICAL WAVE

$t = t_o$

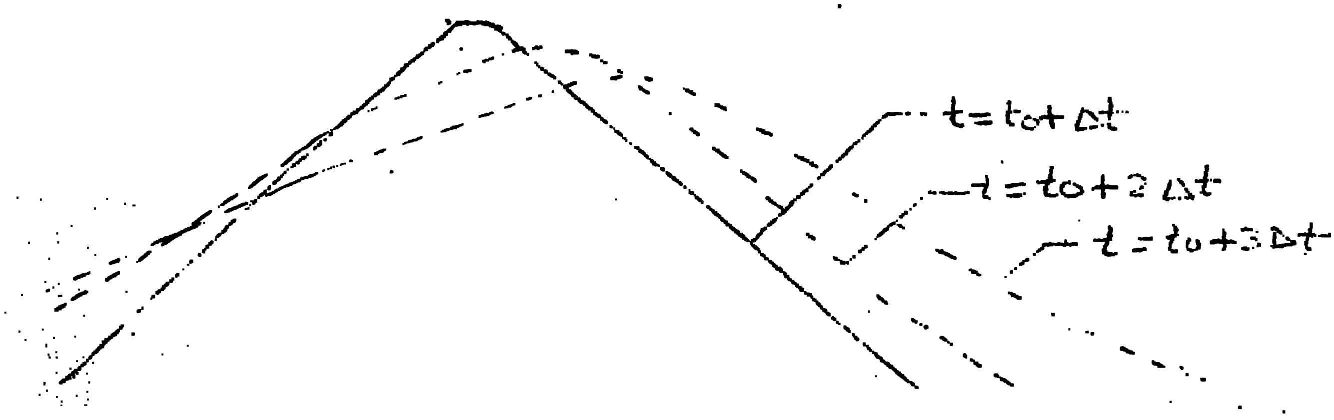


Fig 5·1

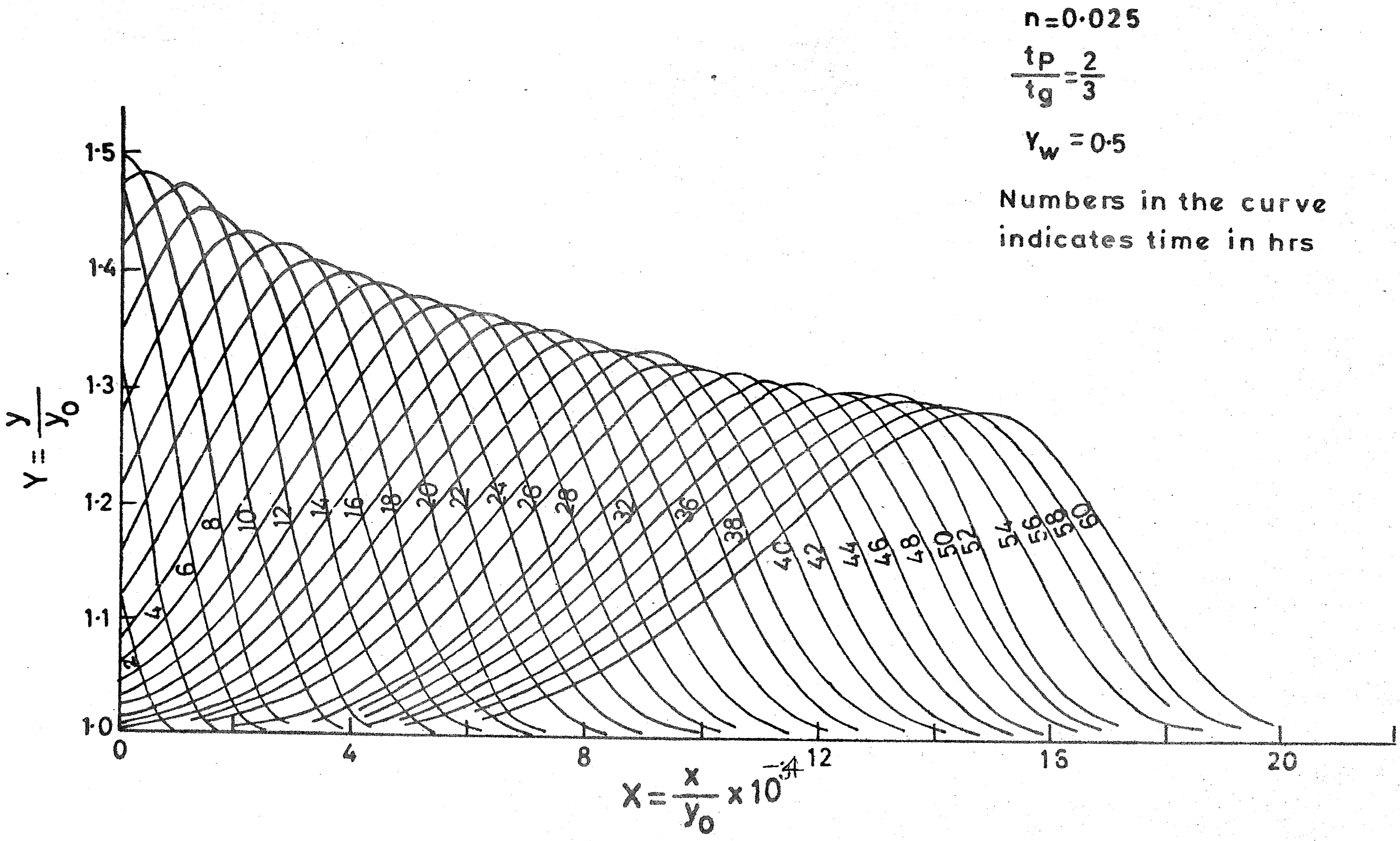


FIG.5·2 SPATIAL PICTURE OF SUBSIDENCE WITH TIME

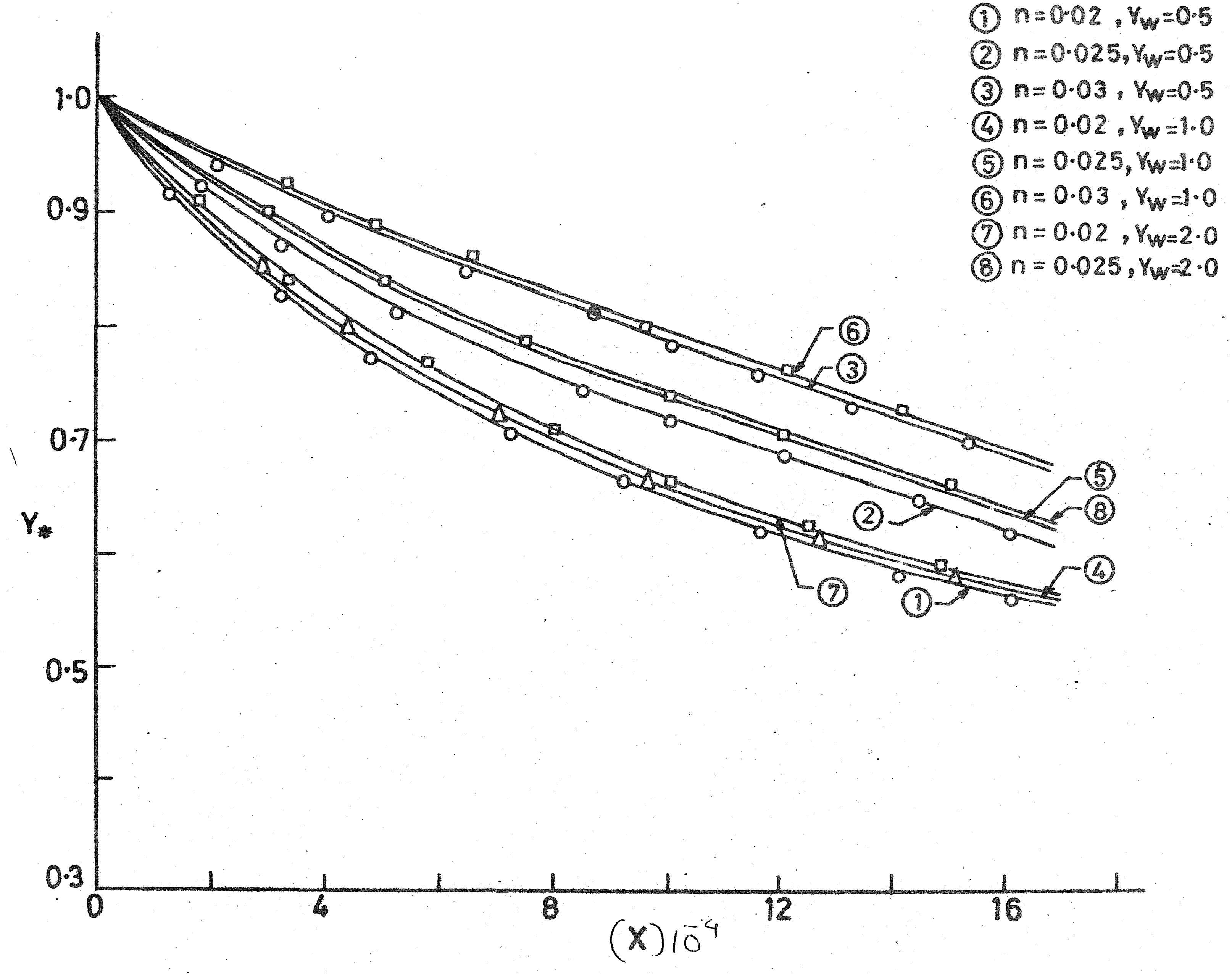


FIG. 5·3 EFFECT OF n AND Y_w ON SUBSIDENCE OF FLOOD WAVE

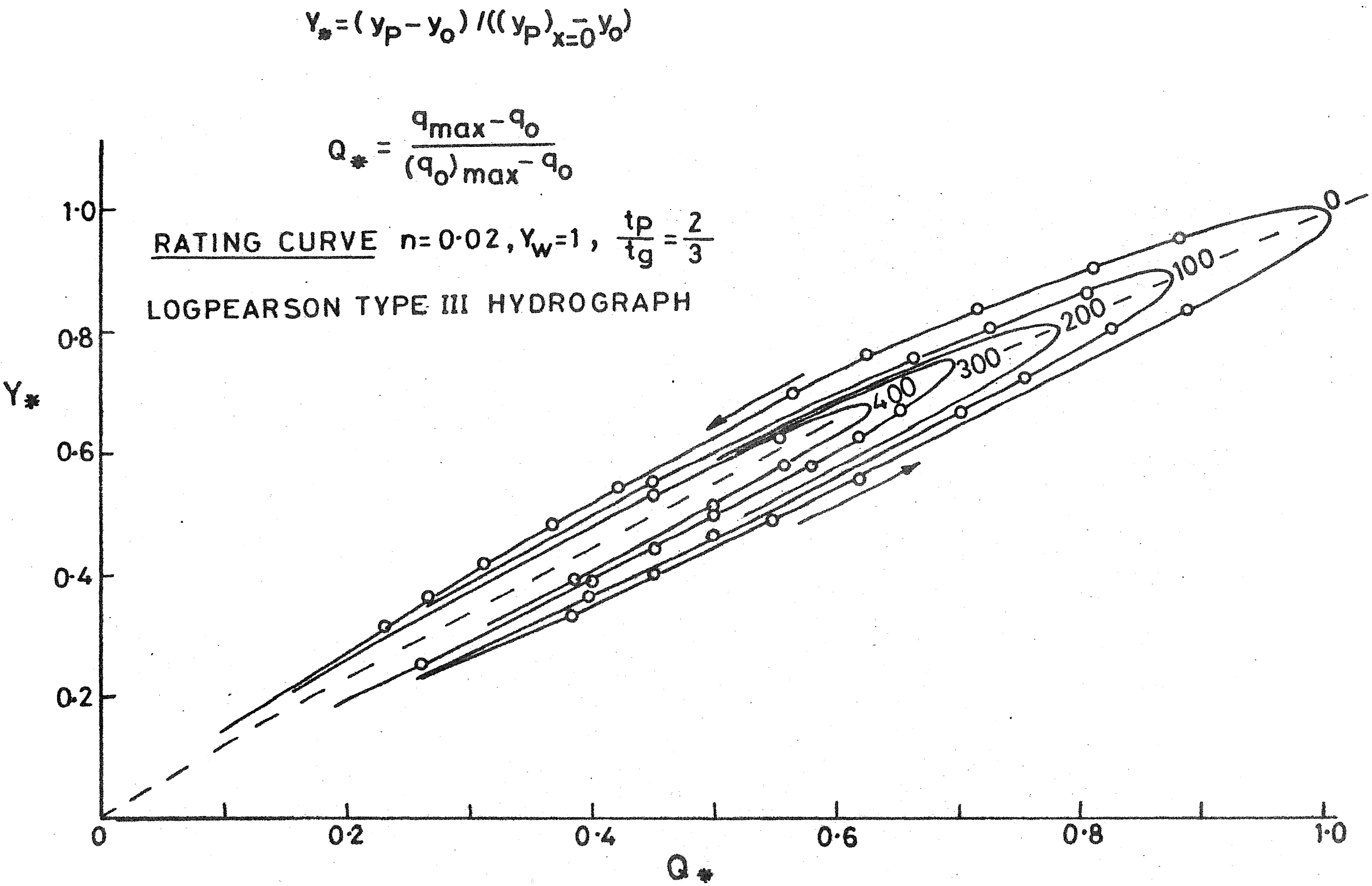


FIG. 5·3 a LOOP RATING CURVES

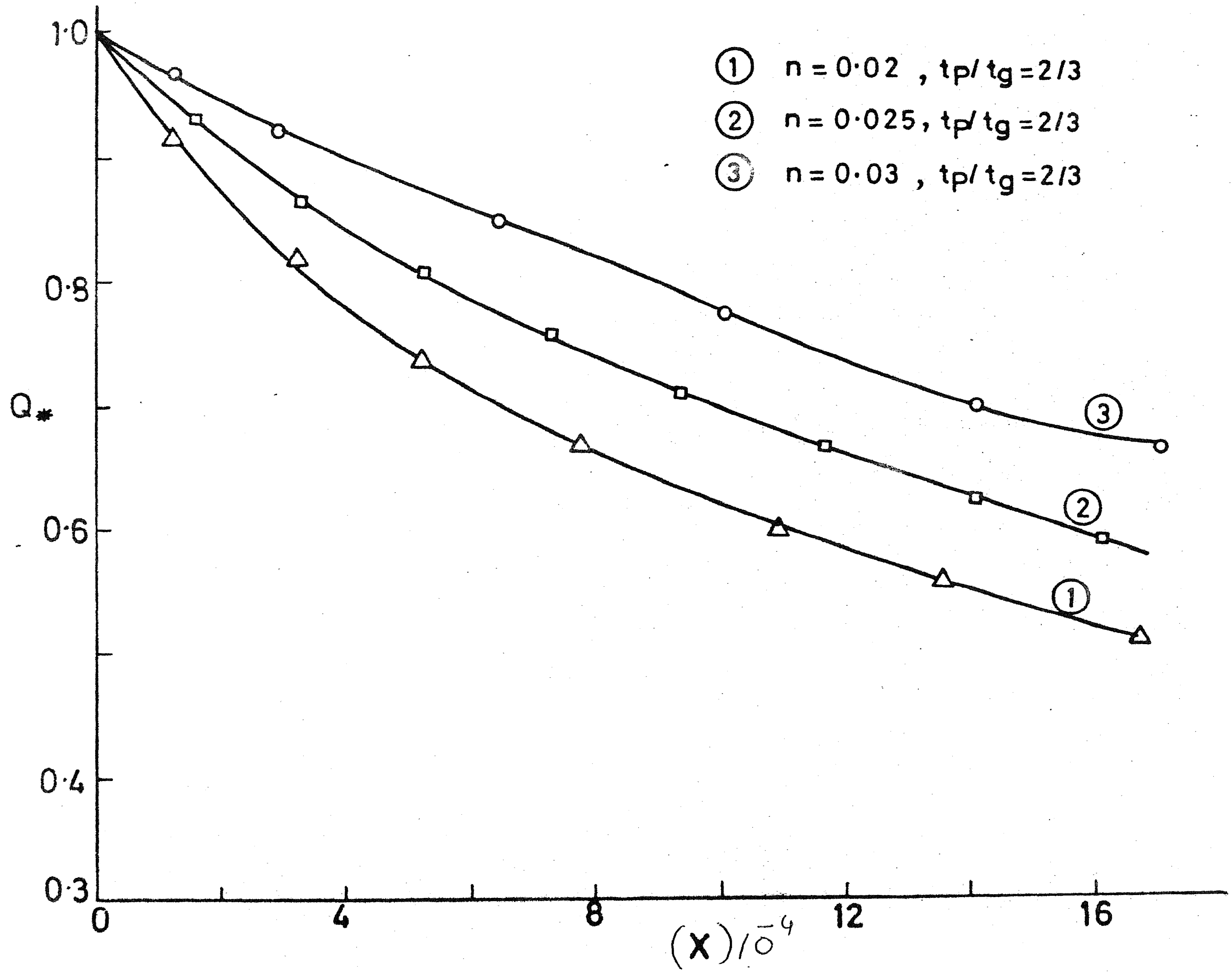


FIG. 5·4 EFFECT OF n ON Q_*

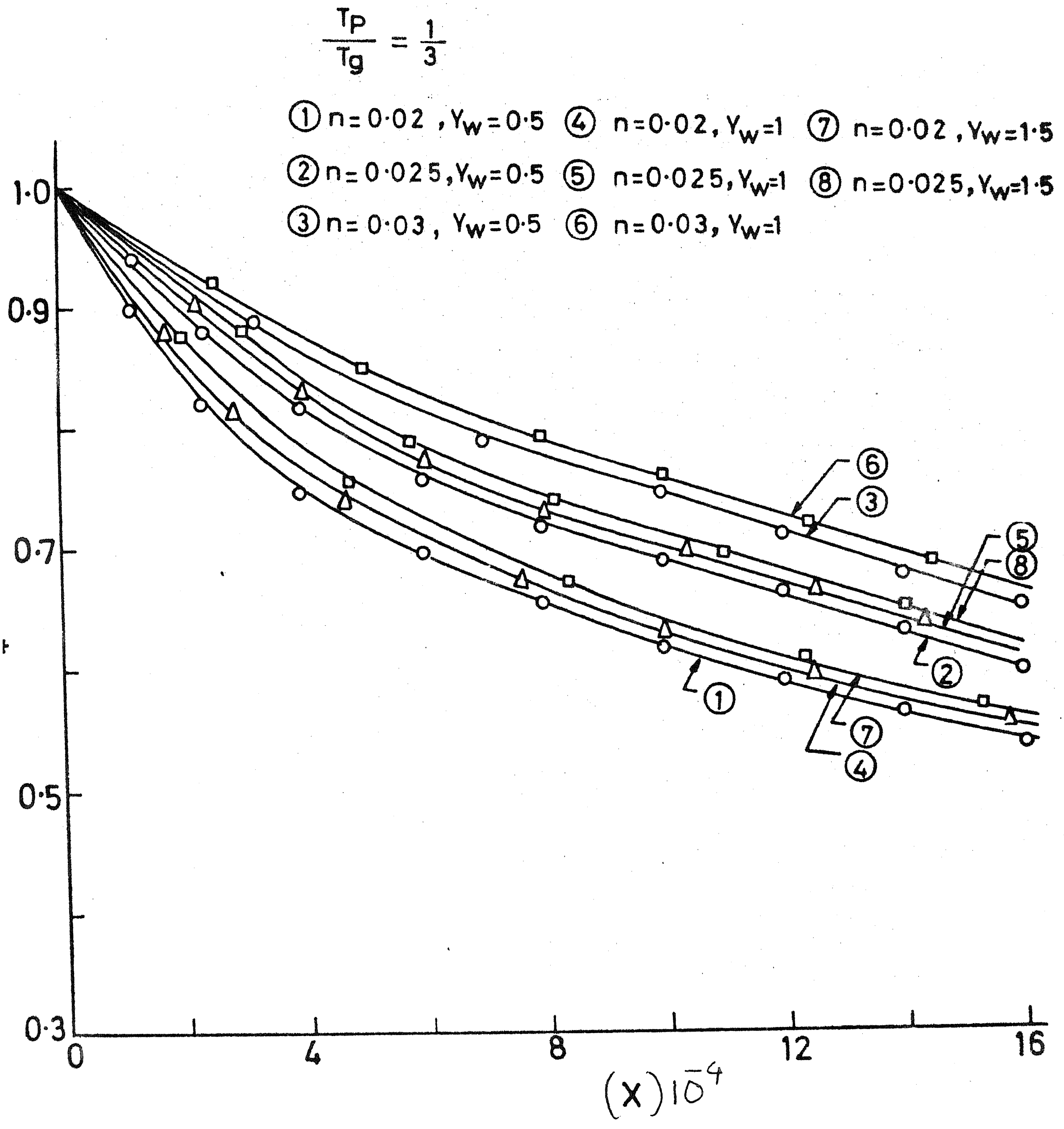


FIG. 5·5 EFFECT OF n AND Y_w ON SUBSIDENCE OF FLOOD WAVE

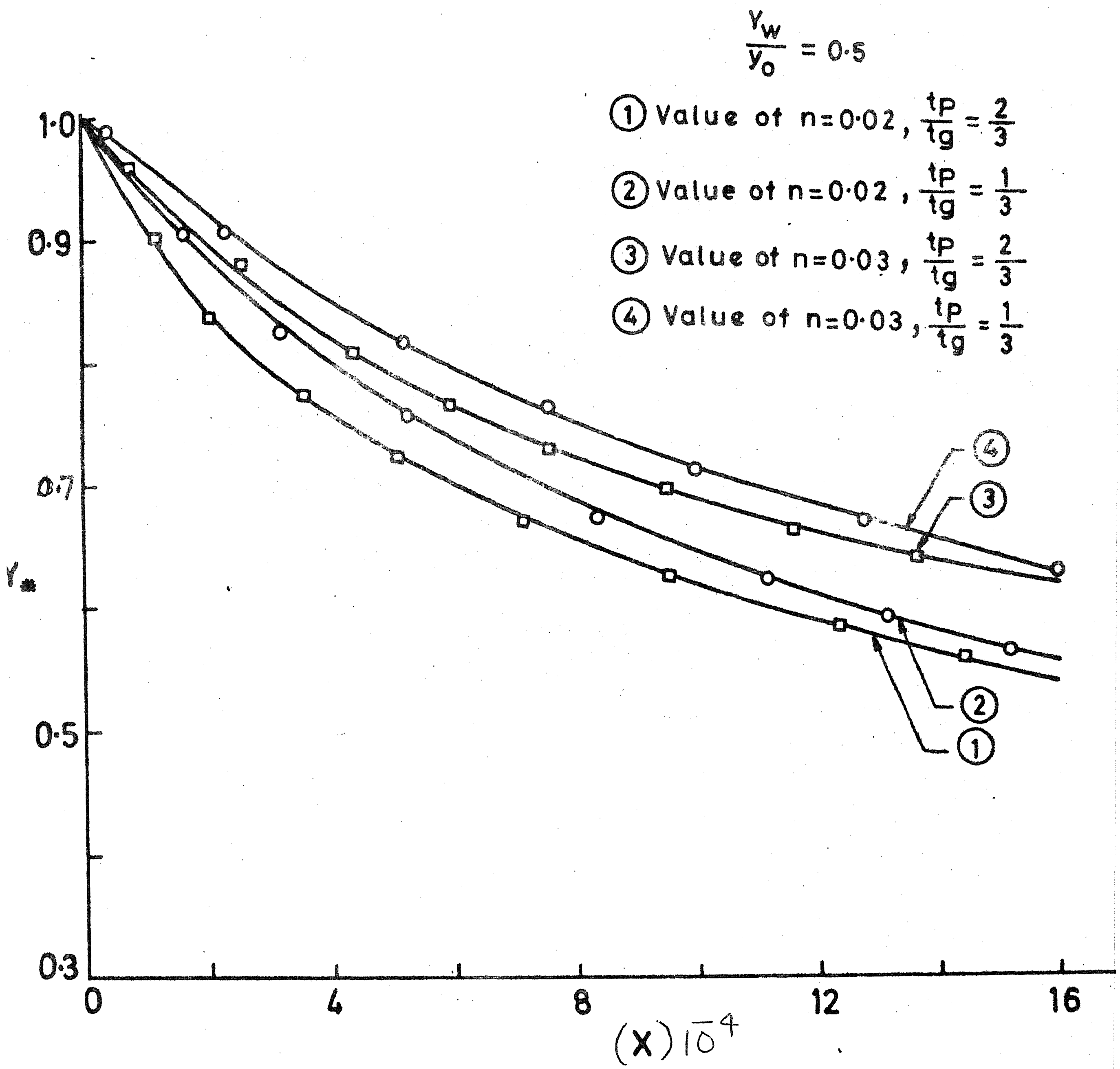


FIG.5·6 EFFECT OF n AND t_p/t_g ON SUBSIDENCE OF FLOOD WAVE

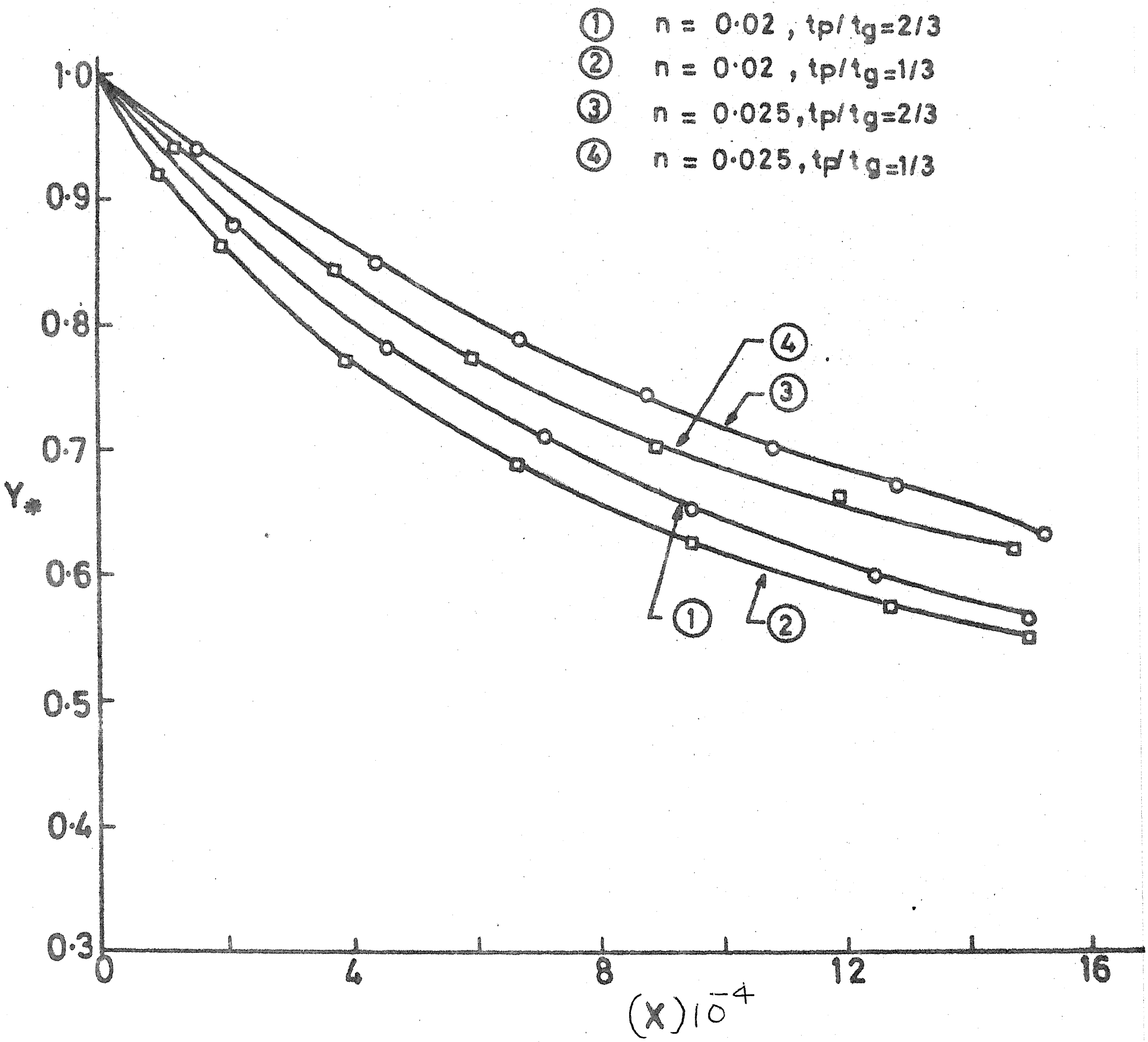


FIG. 5·7 EFFECT OF n AND tp/ tg ON SUBSIDENCE OF FLOOD WAVE

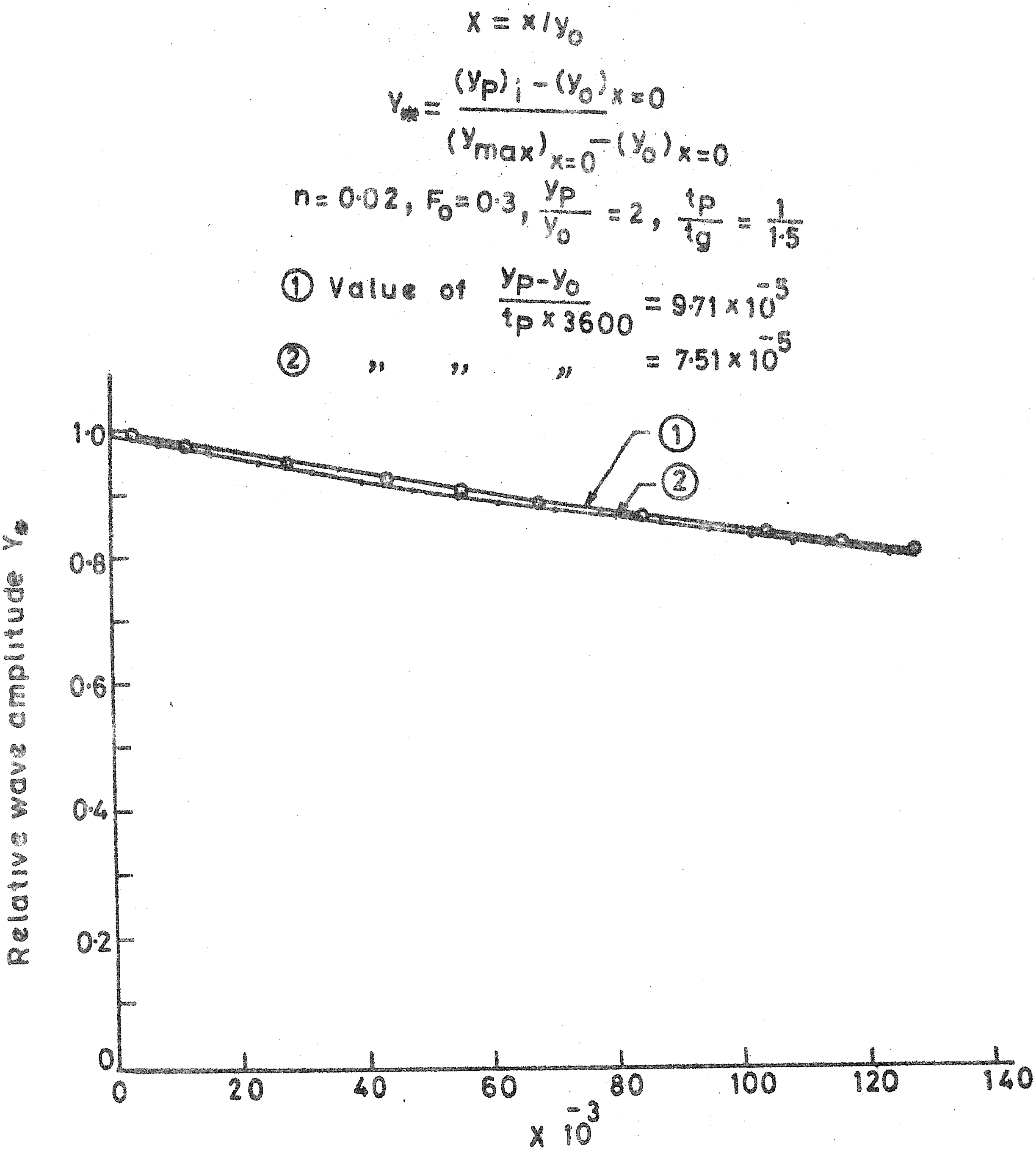


FIG. 5·8 EFFECT OF RATE OF RISE OF HYDROGRA
ON FLOOD WAVE SUBSIDENCE

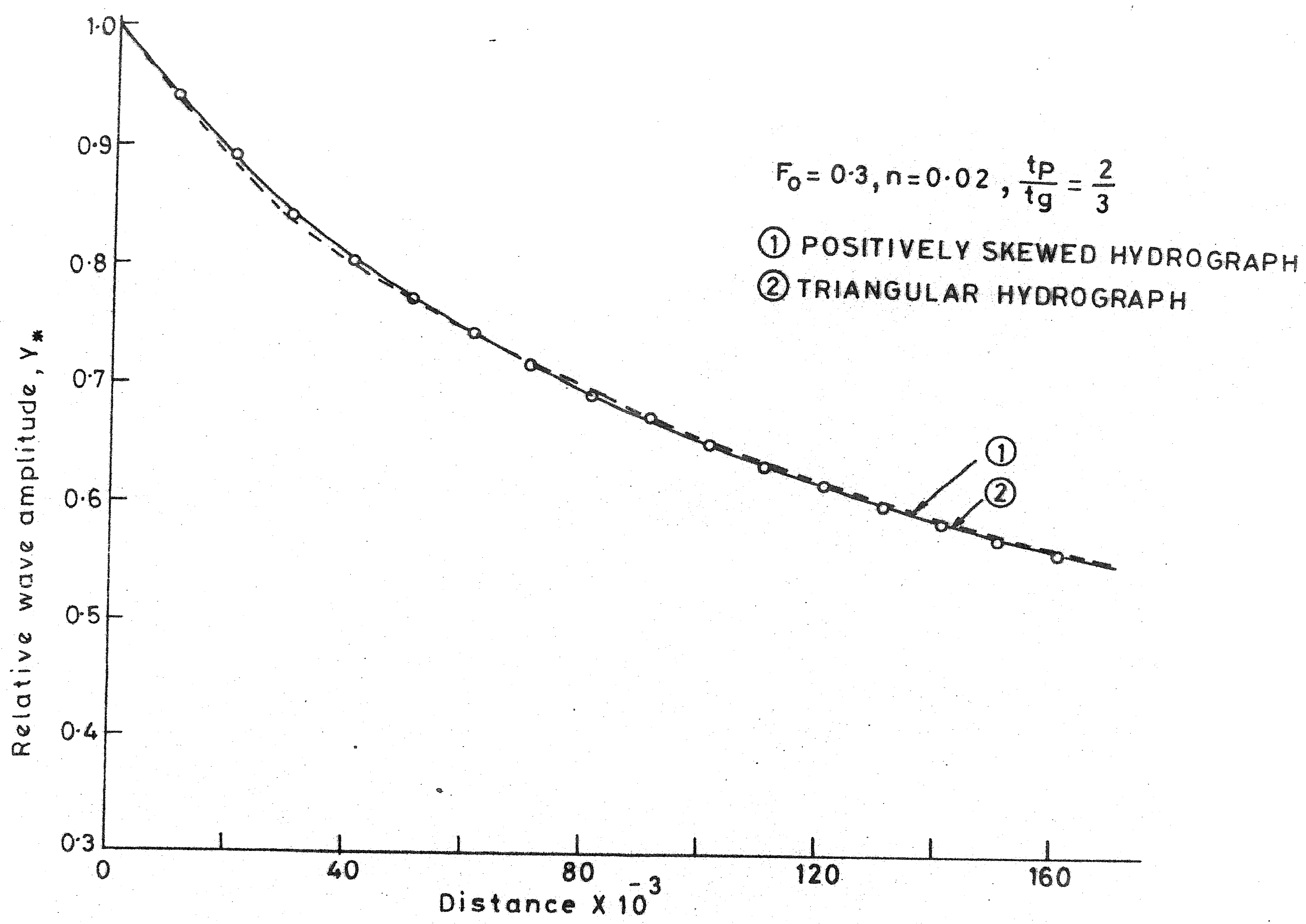


FIG.5·9 EFFECT OF SHAPE OF THE HYDROGRAPH

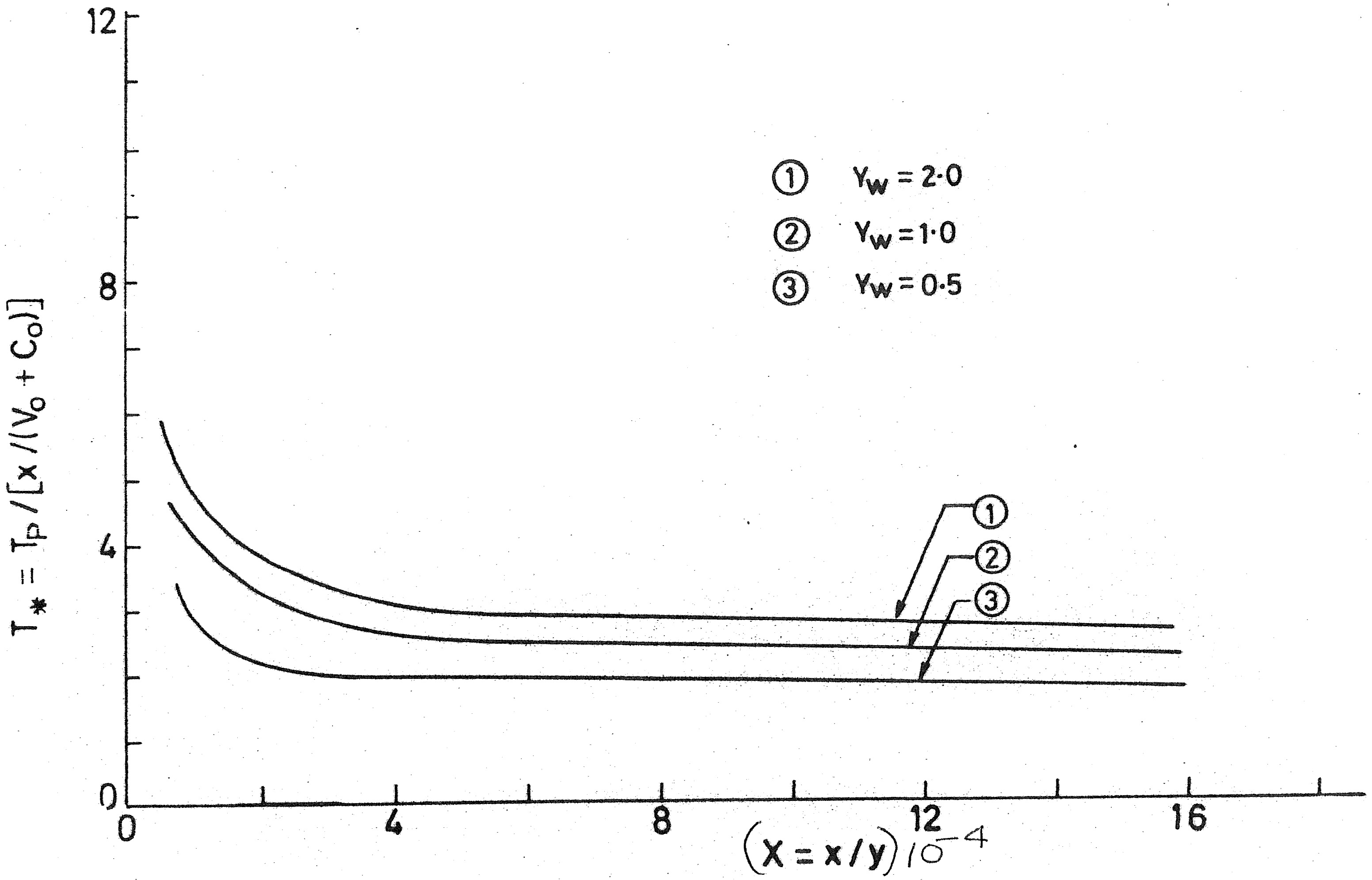


FIG. 5·10 EFFECT OF AMPLITUDE OF FLOOD WAVE ON THE TRAVEL TIME FOR $T_p / T_g = 1/3$

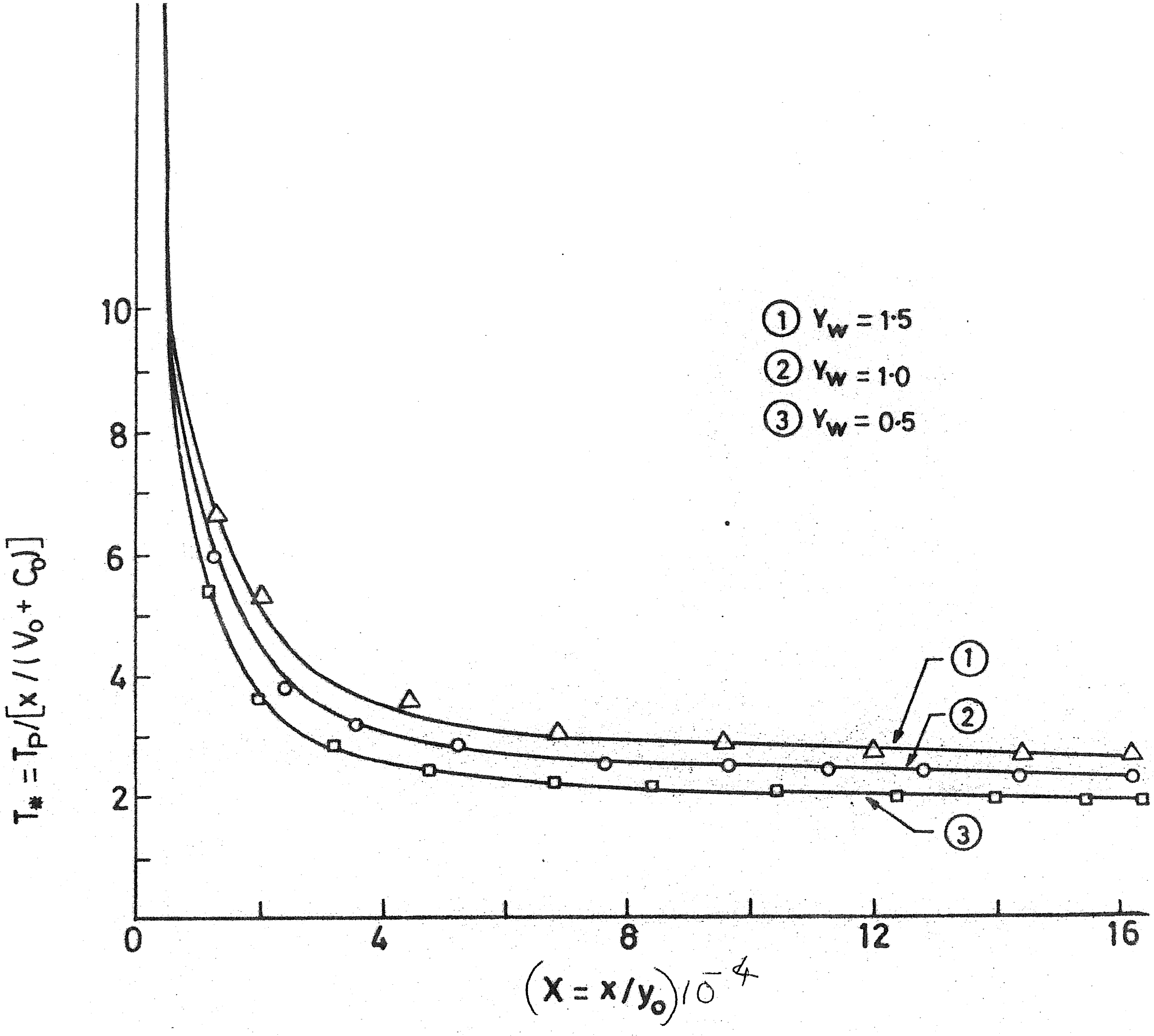
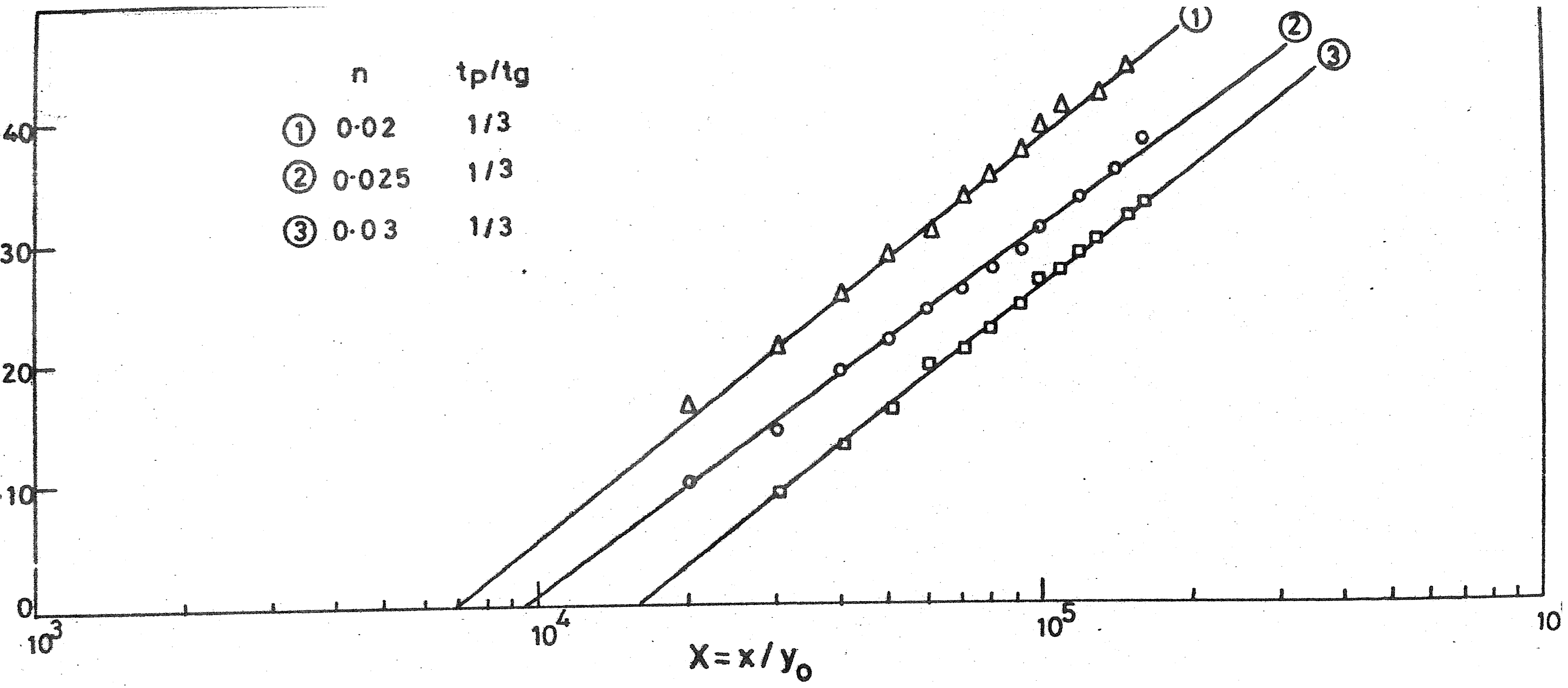
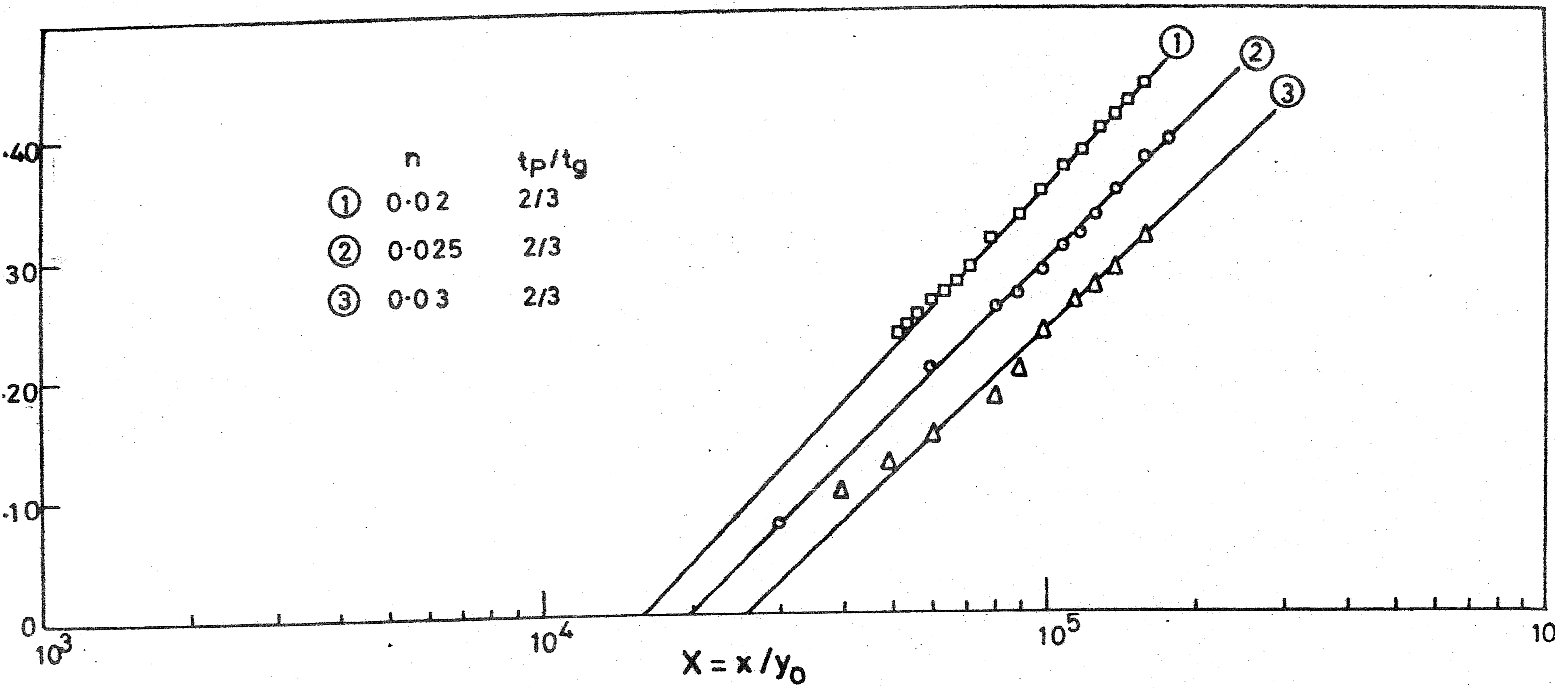


FIG. 5.11 EFFECT OF AMPLITUDE OF FLOOD WAVE ON TRAVEL TIME FOR $T_P / T_a = 2/3$



FIG·6·1 a SUBSIDENCE OF FLOOD WAVE WITH DISTANCE



FIG·6·1b SUBSIDENCE OF FLOOD WAVE WITH DISTANCE

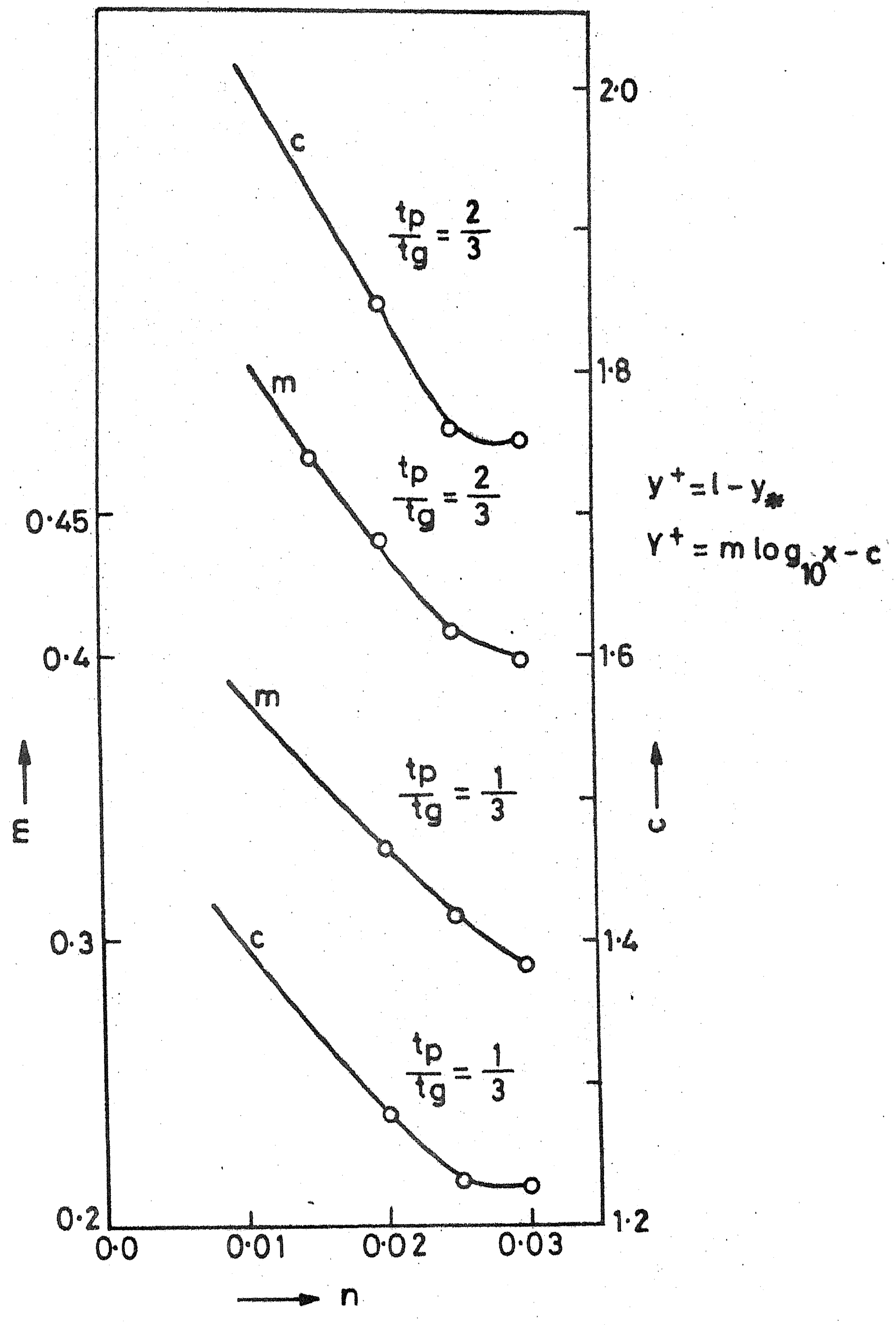


FIG. 6·2 EMPIRICAL CONSTANTS IN THE EQUATION FOR THE DETERMINATION OF FLOOD WAVE SUBSIDENCE

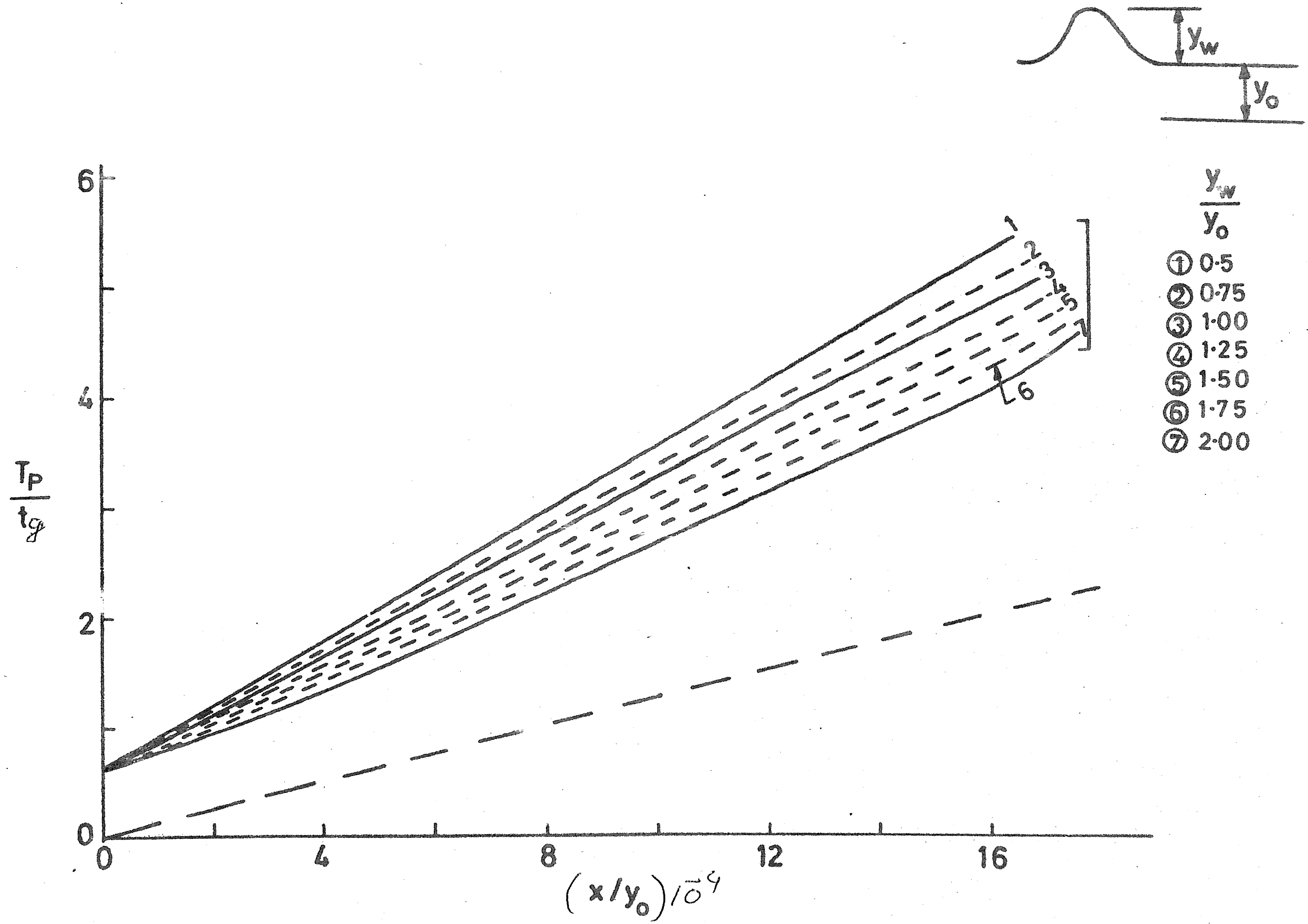


FIG. 6·3 TIME OF OCCURENCE OF FLOOD PEAK VERSUS DISTANCE

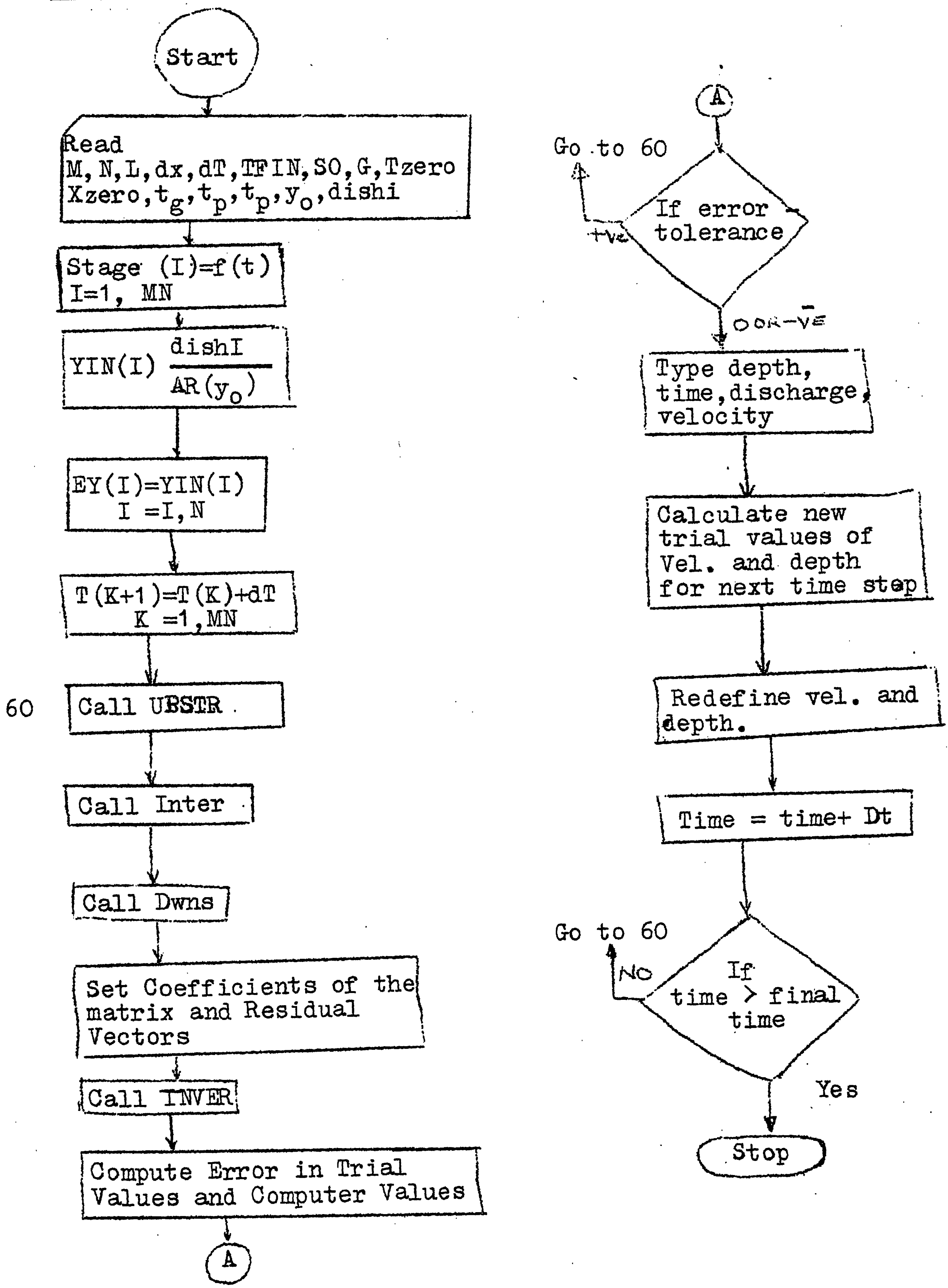
APPENDIX

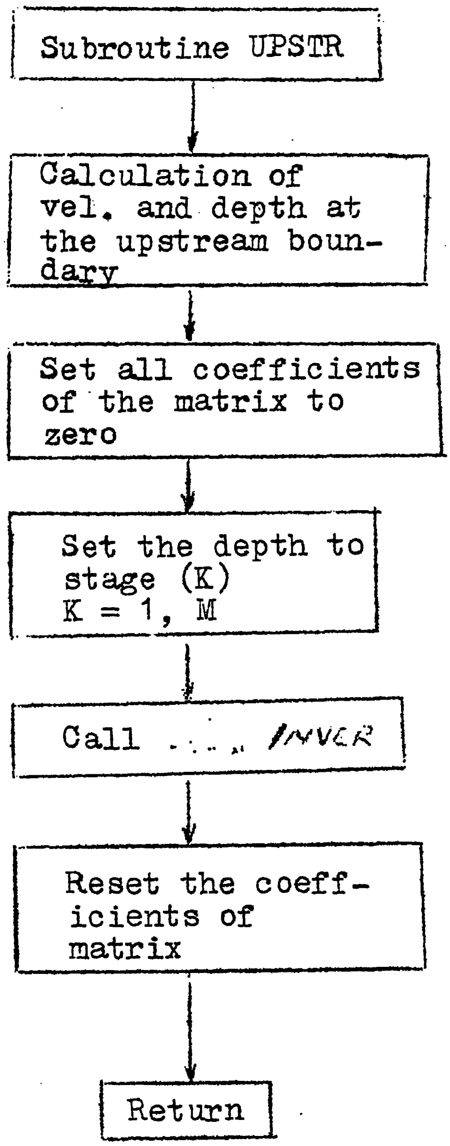
FLOW CHARTS

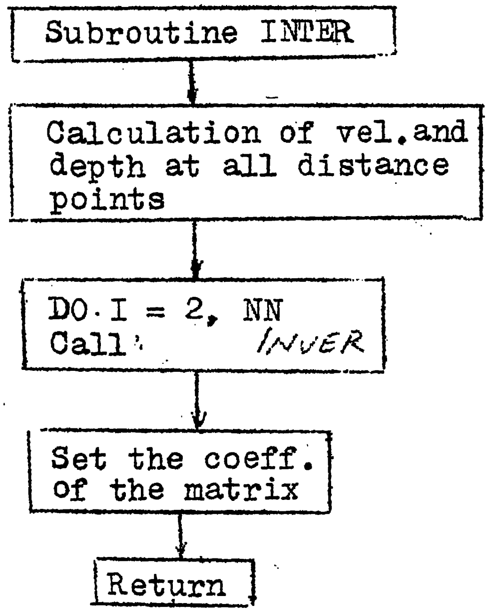
IMPLICIT METHOD OF FLOOD ROUTING
THROUGH IRREGULAR CHANNEL SYSTEMS

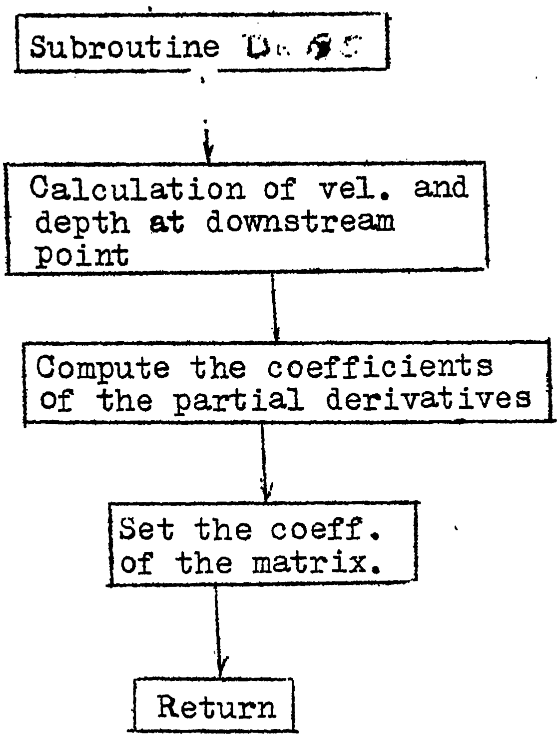
FLOW CHART

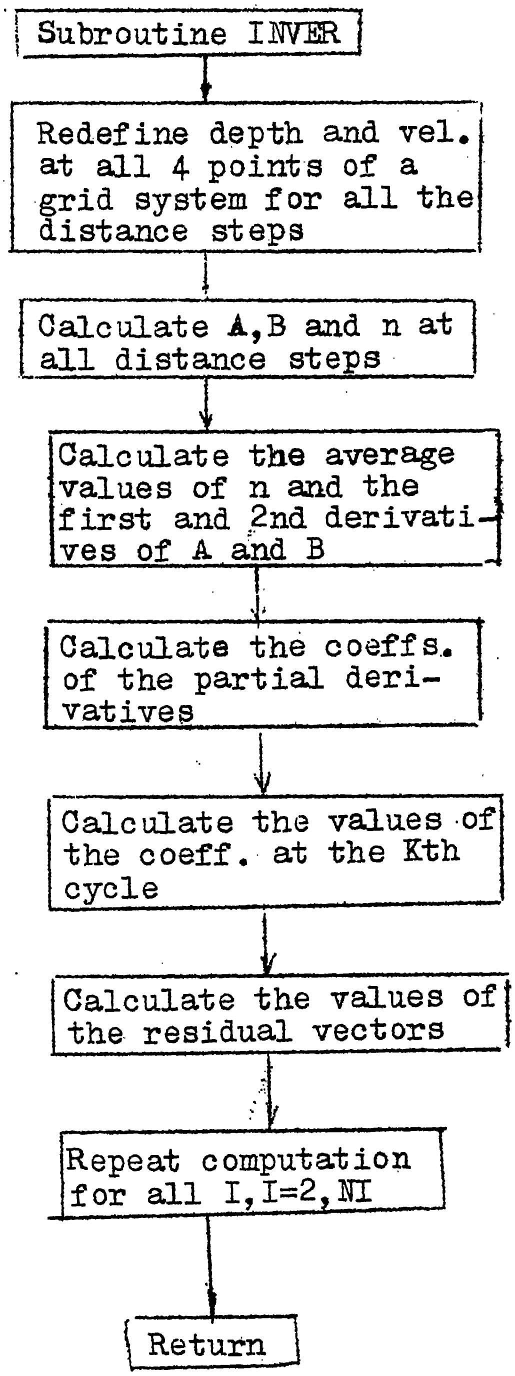
Main Program




Subroutine UPSTR
Calculation of vel. and depth at the upstream boundary
Set all coefficients of the matrix to zero
Set the depth to stage (K) K = 1, M
Call INVER
Reset the coefficients of matrix
Return


Subroutine INTER
Calculation of vel.and depth at all distance points
DO·I = 2, NN
Call INVER
Set the coeff. of the matrix
Return


Subroutine D
Calculation of vel. and depth at downstream point
Compute the coefficients of the partial derivatives
Set the coeff. of the matrix.
Return


Subroutine INVER
Redefine depth and vel. at all 4 points of a grid system for all the distance steps
Calculate A,B and n at all distance steps
Calculate the average values of n and the first and 2nd derivatives of A and B
Calculate the coeffs. of the partial derivatives
Calculate the values of the coeff. at the Kth cycle
Calculate the values of the residual vectors
Repeat computation for all I,I=2,NI
Return

APPENDIX

IMPLICIT METHOD OF FLOOD ROUTING FOR PARAMETRIC STUDY IN A RECTANGULAR CHANNEL

```
!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
   TERMS ARE EXPLAINED HERE
!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
A(I,J)   =MATRIX OF COEFFIENTS
DELX     =INCRIMENT OF DISTANCE IN KILOMETERS
G        =GRAVITATIONAL CONSTANTS
XFIN     =FINAL TIME
INVER    =A SUBROUTINE FOR SOLVING THE BANDED MATRIX
DISHI    =INITIAL DIS. AT U/S END IN CUBIC METERS/SEC.
DISHO    =INITIAL DIS. AT D/S END IN CUBIC METERS/SEC.
SQ       =LATERAL INFLOW IN CUBIC METERS/SEC.PER METER LENGTH
DI       =INITIAL DEPTH AT U/S END IN METERS
DN       =INITIAL DEPTH AT D/S END IN METERS
NL       =NO. OF EQUATIONS AND NO. OF UNKNOWNS
TIME     =TIME AT ANY INSTANT IN HOURS.
T(K)     =VALUE OF T ON T-AXIS AT EACH GIVEN POINT OF MN IN HOURS
C(K)     =VECTOR OF RESIDUES
SO       =CHANNEL BOTTOM SLOPE
DIST(K)  =DISTANCE FOR EACH GRID POINTS FROM X=0 IN KMS.
YIN(K)   =DEPTH AT T=J
VIN(K)   =VELOCTY AT T=J
EV(K)    =VELOCTY AT T=J+1
X(K)     =VECTOR OF NEW TRIAL VALUES
STAGE(K)=INPUT HYDROGRAPH GIVEN IN DTIME,IN METERS
DG(K)    =INTERPOLATION FUNCTION IN TERMS OF DISTANCE
DISCH(K)=DISCHARGE IN CUBIC METERS/SEC.
EA(K)    =AREA AT EACH GRID POINT BASED ON DEPTH.IN SQ.METERS
Q(K)     =LATERAL INFLOW =SQ
EQ(K)    =DISCHARGE  AT EACH GRID POINT BASED ON DEPTH IN M/SEC.
MN       =NO. OF POINTS GIVEN ON T-AXIS
N        =NO. OF POINTS GIVEN ON X-AXIS
L        =INDEX FOR CURRENT VALUE ON THE T-AXIS
TZERO    =INITIAL TIME IN HOURS
DTIME    =TIME INTERVAL BETWEEN DISCRETE POINTS ON T-AXIS
XZERO    =INITIAL DISTANCE IN KMS.
AR       =X-SECTIONAL AREA EQUIATIONS
BR       =TOP WIDTH FUNCTION
DAY      =FIRST DERIVATIVE OF AREA W.R.T DEPTH Y
DDAY     =SECOND DERIVATIVE OF AREA FUNCTION
DBY      =FIRST DERIVATIVE OF TOP WIDTH
YO        =INITIAL STEADY FLOW DEPTH
YW        =INITIAL WAVE AMPLITUDE
TG        =TIME TO THE CENTRE OF GRAVITY OF THE HYDROGRAPH
    TP          =TIME TO OCCURRENCE OF PEAK IN THE INFLOW HYDROGRAPH

                   MAIN PROGRAMME

      COMMON/ABLOK/Z(150),DIST(150),YIN(150),VIN(150),EV(150),EY(150),
     2DG(150),DISCH(150),T(150),VT(150),YT(150),Q(150),EQ(150),
     3DY(150),DV(150),STAGE(150),EA(150)
      COMMON/BBLOK/G,DTIME,SO,DTDX,DXDT,DX,DT,TIME,DR1,DR2,S,R,
     2V,U,DR1DU,DR1DV,DR1DS,DR1DT,DR2DU,DR2DV,DR2DS,DR2DT
      COMMON/CBLOK/MN,N,L,NL,MC,I,MM
      COMMON/DBLOK/SO,ELEVO,ELEVN,XFIN,DELX,DELT,XZERO,DFIN
      COMMON/FBLOK/A(265,265),C1(265),C(265),X(265)
      DIMENSION B(31,31)
      TYPE 1
      OPEN(UNIT=21,FILE='NEW.CDR')
         INITIAL DATA
      ---------------
@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@
THIS PROGRAMME IS WITH THETA=0.5.
@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@
```

```
      TP=0.0
      READ(21,*),MN,N,L
      READ(21,*),TFIN,TZERO,G,DTIME,XZERO,DISHI,DISHN,
     2DELX,XFIN,DELT,YW,YO,SO,TP,TG
      DO 10 I=1,N
10    STAGE(I)=YO+YW*EXP(-((2*(I-1))-TP)/(TG-TP))*(((2*(I-1)))/TP)**
     1(TP/(TG-TP))
      TYPE 29,(STAGE(K),K=1,MN)
29    FORMAT(5X,'INFLOW HYDROGRAPH IS',/5X,20F6.2/10F6.2)
      DN           =YO
      DI           =STAGE(1)
      VI           =DISHI/AR(DI)
      VN           =DISHN/AR(DN)
      SO           =(DISHN-DISHI)/(XFIN*1000.0)
      TYPE 9
9     FORMAT(3X,'INFLOW HYDROGRAPH IS LOGPEARSON TYPE 3',/3X,42(1H-))
      CL=CI(DI)
      TYPE 7,SO,DISHI,YO,YW,TG,CL,TP
7     FORMAT(5X,'SLOPE= ',E15.7,'QO= ',F5.2,'YO= ',F5.2,'YW= ',F5.2,
     2'TG= ',F5.2,'MANNING"S N=',E15.7,'TP= ',F5.2,/5X,80(1H-))
      TYPE 29
29    FORMAT(/70(1H))
         TYPE 21
      MM           =MN-1
      NL           =2*N-1
      NN           =N-1
      T(1)         =TZERO
      DIST(1)      =0.0
      ITRAY=1
      DO 30 I      =1,NN
      DIST(I+1)    =DIST(I)+DELX
30    CONTINUE
      DT=DELT*60.0*60.0
      DX           =DELX*1000.0
      DTDX         =DT/DX
      DXDT         =DX/DT
      TIME         =TZERO
      DO 50 I      =1,N
      DG(I)        =(DIST(I)-XZERO)/(XFIN-XZERO)
      VIN(I)       =VI+(VN-VI)*DG(I)
      YIN(I)       =DI+(DN-DI)*DG(I)
      TYPE 39,I,VIN(I),I,YIN(I),I,DG(I)
39    FORMAT(/,5X,'VIN(',I3,')=',F10.4,5X,'YIN(',I3,')=',F10.4,
     25X,'DG(',I3,')=',F10.4)
      EV(I)        =VIN(I)
      EY(I)        =YIN(I)
      DISCH(I)     =DISHI+(DISHN-DISHI)*DG(I)
      Q(I)         =SQ
50    CONTINUE
      DO 60 K      =1,MM
60    T(K+1)       =T(K)+DTIME
      TIME         =TIME+DELT
65    CALL UPSTR
      CALL INTER
      CALL DWNS
      DO 180 I     =1,NL
180   C1(I)        =C(I)
13    FORMAT(1X,16F5.1)
      DO 177 M=1,NL
      DO 177 J=1,NL
177   B(M,J)=A(M,J)
      CALL INVER(B,C1,NL,KS)
      DO 190 I     =1,NL
      X(I)         =C1(I)
190   CONTINUE
      DO 200 I     =1,N
      VI(I)        =EV(I)
```

```
200 YT(I)       =EY(I)
    K           =NL-1
    DO 210 J    =1,K,2
    I           =(J+1)/2
    EV(I)       =X(J)
210 EY(I+1)     =X(J+1)
    EV(N)       =X(NL)
    ITRAY=ITRAY+1
    IF(ITRAY-50)330,330,335
335 TYPE 333,ITRAY,EY(1),EY(N),EV(1),EV(N)
333 FORMAT(1H ,I2,4E11.3)
    CALL EXIT
330 CONTINUE
    DO 240 I    =1,N
    ERROR       =(EV(I)-VT(I))/EV(I)*100.0
    IF(ABS(ERROR)-2.0) 240,240,65
240 CONTINUE
    IF(IP-TIME) 245,245,255
245 CONTINUE
    IP=IP+2.0
    TYPE 31,TIME
    TYPE 23
31  FORMAT(5X,F7.2)
    DO 250 I    =1,N
    EA(I)=AR(EY(I))
    EQ(I)=EA(I)*EV(I)
250 TYPE 25,DIST(I),EY(I),EQ(I)
    DO 260 I    =1,N
255 CONTINUE
    ITRAY=1
    DY(I)       =EY(I)-YIN(I)
260 DV(I)       =EV(I)-VIN(I)
    DO 270 I    =1,N
    YIN(I)      =EY(I)
270 VIN(I)      =EV(I)
    TIME        =TIME+DELT
    IF(2-TIME) 300,280,280
280 DO 285 I    =1,N
    EY(I)       =EY(I)+DY(I)*.5
285 EV(I)       =EV(I)+DV(I)*.5
    GO TO 65
1   FORMAT(35X,'STREAMFLOW ROUTING BY IMPLICIT METHOD'/33X,
   241(1H*))
12  FORMAT(5X,11(2X,F9.4))
21  FORMAT(15X,'VALUES OF THE VARIABLES')
23  FORMAT(6X,'DISTANCE',3X,'DEPTH',5X,'DISCHARGE')
25  FORMAT(6X,F6.1,4X,F5.2,4X,F10.3)
300 CALL EXIT
    END
C
        SUBROUTINE UPSTR
C       ------------------------
        COMMON/ABLOK/Z(150),DIST(150),YIN(150),VIN(150),EV(150),EY(15(
   20G(150),DISCH(150),T(150),VT(150),YT(150),Q(150),EQ(150),
   3DY(150),DV(150),STAGE(150),EA(150)
    COMMON/BBLOK/G,DTIME,SO,DTDX,DXDT,DX,DT,TIME,DR1,DR2,S,R,
   2V,U,DR1DU,DR1DV,DR1DS,DF1DT,DR2DU,DR2DV,DR2DS,DR2DT
    COMMON/CBLOK/NN,N,L,NL,MN,I,MM
    COMMON/DBLOK/SO,ELEVO,ELEVN,XFIN,DELX,DELT,XZERO,DFIN
    COMMON/FBLOK/A(265,265),C1(265),C(265),X(265)
C
C       COMPUTATION AT U/S BOUNDARY
C       ------------------------------
65  DO 70 I     =1,NL
    X(I)        =0.0
    DO 70 J     =1,NL
70  A(I,J)      =0.0
```

```
      IF(TIME-T(L)) 74,71,72
71    SURFL     =STAGE(L)
      GO TO 80
72    L         =L+1
74    SURFL     =STAGE(L-1)+(TIME-T(L-1))*(STAGE(L)-STAGE(L-1))/DTIME
80    EY(1)     =SURFL
      I         =1
      CALL INVER
      A(1,1)    =DR1DU
      A(1,2)    =DR1DT
      A(1,3)    =DR1DV
      A(2,1)    =DR2DU
      A(2,2)    =DR2DT
      A(2,3)    =DR2DV
C     TYPE 14,DR1DU,DR1DT,DR1DV,DR2DU,DR2DT,DR2DV
14    FORMAT(2X,'A(1,1)='1X,F5.2,1X,'A(1,2)=',1X,F5.2,1X,'A(1,3)=',1X,
C     1F5.2,'A(2,1)='F5.2,'A(2,2)='F5.2,2X,'A(2,3)='F5.2/80(1H-))
      C(1)      =DR1DU*U+DR1DV*V+DR1DT*R-DR1
      C(2)      =DR2DU*U+DR2DV*V+DR2DT*R-DR2
C     TYPE 24,C(1),C(2)
24    FORMAT(3X,'C(1)=',F5.1,2X,'C(2)=',F5.1/20(1H-))
      RETURN
      END
C
          SUBROUTINE INTER
C         -------------------
          COMMON/ABLOK/Z(150),DIST(150),YIN(150),VIN(150),EV(150),EY(150
     2DG(150),DISCH(150),T(150),VT(150),YT(150),Q(150),EQ(150),
     3DY(150),DV(150),STAGE(150),EA(150)
      COMMON/BBLOK/G,DTIME,SO,DTDX,DXDT,DX,DT,TIME,DR1,DR2,S,R,
     2V,U,DR1DU,DR1DV,DR1DS,DR1DT,DR2DU,DR2DV,DR2DS,DR2DT
      COMMON/CBLOK/NO,N,L,ND,MN,I,MM
      COMMON/DBLOK/SO,ELEVO,ELEVN,XFIN,DELX,DELT,XZERO,DFIN
      COMMON/FBLOK/A(265,265),C1(265),C(265),X(265)
C     TYPE 11
11    FORMAT(5X,'C(M),C(M+1),A(M,J),A(M,J+1),A(M,J+2),A(M,J+3),
     1A(M+1,J),A(M+1,J+1),A(M+1,J+2),A(M+1,J+3) ARE AS FOLLOWS')
C         COMPUTATION AT INTERIOR POINTS
C         ---------------------------------
100   DO 150 I  =2,MN
      CALL INVER
      J         =2*I-2
      M         =2*I-1
      A(M,J)    =DR1DS
      A(M,J+1)  =DR1DU
      A(M,J+2)  =DR1DT
      A(M,J+3)  =DR1DV
      A(M+1,J)  =DR2DS
      A(M+1,J+1)=DR2DU
      A(M+1,J+2)=DR2DT
      A(M+1,J+3)=DR2DV
      C(M)      =DR1DU*U+DR1DV*V+DR1DT*R-DR1+DR1DS*S
      C(M+1)    =DR2DU*U+DR2DV*V+DR2DT*R-DR2+DR2DS*S
C     TYPE 23,C(M),C(M+1)
23    FORMAT(2X,2F5.1/)
      TYPE 22,A(M,J),A(M,J+1),A(M,J+2),A(M,J+3),A(M+1,J),A(M+1,J+1),
     1A(M+1,J+2),A(M+1,J+3)
C     TYPE 11
22    FORMAT(2X,6F5.1)
150   CONTINUE
      RETURN
      END
C
C
C     THIS SUBROUTINE IS FOR CALCULATIONS AT DOWNSTREM BOUNDARY.
C
          SUBROUTINE DWBS
C         -------------------
```

```
      COMMON/ABLOK/Z(150),DIST(150),YIN(150),VIN(150),EV(150),EY(150)
     2DG(150),DISCH(150),T(150),VT(150),YT(150),Q(150),EQ(150),
     3DY(150),DV(150),STAGE(150),EA(150)
      COMMON/BBLOK/G,DTIME,SO,DTDX,DXDT,DX,DT,TIME,DR1,DR2,S,R,
     2V,U,DR1DU,DR1DV,DR1DS,DR1DT,DR2DU,DR2DV,DR2DS,DR2DT
      COMMON/CBLOK/NN,N,L,NL,MN,I,MM
      COMMON/DBLOK/SO,ELEVO,ELEVN,XFIN,DELX,DELT,XZERO,DFIN
      COMMON/FBLOK/A(265,265),C1(265),C(265),X(265)

         COMPUTATION AT U/S BOUNDARY
        -------------------------------
      P              =EV(N)
      R              =EY(N)
      DADY           =DAY(R)
      V1             =P*P
      A1             =AR(R)
      A2             =A1*A1
      W              =P*A1
      CU=CI(R)
      DR1=R-5.0
      DR1DT=1.0
      DR1DV=0.0
      A(NL,NL-1)=DR1DT
      A(NL,NL)   =DR1DV
      C(NL)      =DR1DT*R+DR1DV*P-DR1
      RETURN
      END
      $$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$
      THIS SUBROUTINE CALCULATES THE COEFFICIENT AND RESIDUES
      $$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$
         SUBROUTINE INVER
        -------------------
         COMMON/ABLOK/Z(150),DIST(150),YIN(150),VIN(150),EV(150),EY(150
     2DG(150),DISCH(150),T(150),VT(150),YT(150),Q(150),EQ(150),
     3DY(150),DV(150),STAGE(150),EA(150)
      COMMON/BBLOK/G,DTIME,SO,DTDX,DXDT,DX,DT,TIME,DR1,DR2,S,R,
     2V,U,DR1DU,DR1DV,DR1DS,DR1DT,DR2DU,DR2DV,DR2DS,DR2DT
      COMMON/CBLOK/NN,N,L,NL,MN,I,MM
      COMMON/DBLOK/SO,ELEVO,ELEVN,XFIN,DELX,DELT,XZERO,DFIN
      COMMON/FBLOK/A(265,265),C1(265),C(265),X(265)

         SOLUTION OF SYSTEM OF EQUATIONS WITH 4 NONZERO TERMS ON
                          THE BANDED MATRIX
        ---------------------------------------------------------------
      U              =EV(I)
      V              =EV(I+1)
      S              =EY(I)
      R              =EY(I+1)
      TYPE *,S,R
      TYPE 39
   39 FORMAT(10X,'S=    R= ')
      SB             =YIN(I)
      RB             =YIN(I+1)
      UB             =VIN(I)
      VB             =VIN(I+1)
      A1             =AR(S)
      A2             =AR(R)
      B1             =BR(S)
      B2             =BR(R)
      AB1            =AR(SB)
      AB2            =AR(RB)
      BB1            =BR(SB)
      BB2            =BR(RB)
      CB1            =CI(SB)+(CN(SB)-CI(SB))*DG(I)
      CB2            =CI(RB)+(CN(RB)-CI(RB))*DG(I+1)
      CI1            =CI(S)
      CN1            =CN(S)
```

```
CI2          =CI(R)
CN2          =CN(R)
CO1          =CI1+(CN1-CI1)*DG(I)
CO2          =CI2+(CN2-CI2)*DG(I+1)
Q1           =Q(I)
Q2           =Q(I+1)
DA1DY        =DAY(S)
DA2DY        =DAY(R)
DB1DY        =DBY(S)
DB2DY        =DBY(R)
DB3DY        =DDAY(S)
DB4DY        =DDAY(R)
DA3DY        =DAY(SB)
DA4DY        =DAY(RB)
V1           =V+VB-U-UB
V2           =V+U-VB-UB
D1           =R-S+RB-SB
D2           =R+S-SB-RB
VD           =V+U+VB+UB
E1           =DA1DY
E2           =DA2DY
EB1          =DA3DY
EB2          =DA4DY
QD           =Q1*(U/A1+V/A2+UB/AB1+VB/AB2)
BD           =1./E1+1./E2+1./EB1+1./EB2
AD           =A1/E1+A2/E2+AB1/EB1+AB2/EB2
SF1          =CO1*CO1*U*ABS(U)*(S)**(-4.0/3.0)
SF2          =CO2*CO2*V*ABS(V)*(R)**(-4.0/3.0)
SF3          =CB1*CB1*UB*ABS(UB)*(SB)**(-4.0/3.0)
SF4          =CB2*CB2*VB*ABS(VB)*(RB)**(-4.0/3.0)
DSF1         =CO1*CO1*U*(S)**(-4.0/3.0)
DSF2         =CO2*CO2*V*R**(-4.0/3.0)
SF           =SF1+SF2+SF3+SF4
DB4          =DB1DY/E1
DB3          =DB3DY/(E1*E1)
DB2          =DB4DY/(E2*E2)
DB1          =DB2DY/E2
DA4          =DA1DY/A1
DA3          =DA1DY/(A1*A1)
DA2          =DA2DY/(A2*A2)
DA1          =DA2DY/A2
DR1          =.25*DTDX*VD*D1+D2+DTDX*.25*V1*AD-.5*Q1*DT*BD
DR2          =DTDX*V2/G+.25*VD*V1/G+.5*DX*QD/G+D1-DX*2*SO+.5*DX*SF
W            =.25*DTDX
DR1DS        =1-W*VD+V1*(DA1DY/E1-A1*DB3)*W+.5*DT*DB3*Q1
DR1DT        =W*VD+1+W*V1*(DA2DY/E2-A2*DB2)+.5*DT*Q1*DB2
DR1DV        =.25*DTDX*D1+DTDX*AD*.25
DR1DU        =.25*DTDX*D1-DTDX*AD*.25
DR2DU        =(DXDT-.5*(U+UB)+.5*DX*Q1/A1)/G+DX*DSF1
DR2DV        =(DXDT+.5*(V+VB)+.5*DX*Q1/A2)/G+DX*DSF2
DR2DT        =1-.5*DX*Q1*V*DA2/G+DX*(4.0/3.0)*SF2*(DB1-DA1)*.5
DR2DS        =-1-.5*DX*Q1*U*DA3/G+DX*(4.0/3.0)*SF1*(DB4-DA4)*.5
RETURN
END
SUBROUTINE INVER  (A,B,N,KS)
DIMENSION A(1),B(1)
FORWARD SOLUTION
TOL=0.0
KS=0
JJ=-N
DO 65 J=1,N
JY=J+1
JJ=JJ+N+1
BIGA=0.
IT=JJ-J
L1=J+4
IF(L1-N) 10,10,12
```

```
12    L1=N
10    DO 30 I=J,L1
C     SEARCH FOR MAXIMUM COEFFICIENT IN COLUMN
      IJ=IT+I
      IF(ABS(BIGA)-ABS(A(IJ)))  20,30,30
20    BIGA=A(IJ)
      IMAX=I
30    CONTINUE
C     TEST FOR PIVOT LESS THAN TOLERANCE(SINGULAR MATRIX
32    IF(ABS(BIGA)-TOL) 35,35,40
35    KS=1
      RETURN
C     INTERCHANGE ROWS IF NECESSARY
40    I1=J+N*(J-2)
      IT=IMAX-J
      L2=J+4
      IF(L2-N) 42,42,44
44    L2=N
42    DO 50 K=J,L2
      I1=I1+N
      I2=I1+IT
      SAVE=A(I1)
      A(I1)=A(I2)
      A(I2)=SAVE
C     DIVIDE EQUATION BY LEADING COEFFICIENT
C     P 104
50    A(I1)=A(I1)/BIGA
52    SAVE=B(IMAX)
      B(IMAX)=B(J)
      B(J)=SAVE/BIGA
C     ELIMINATE NEXT VARIABLE
      IF(J-N) 55,70,55
55    IQS=N*(J-1)
      L3=JY+4
      IF(L3-N) 57,57,59
59    L3=N
57    DO 65 IX=JY,L3
      IXJ=IQS+IX
      IT=J-IX
      DO 60  JX=JY,L3
      IXJX=N*(JX-1)+IX
      JJX=IXJX+IT
60    A(IXJX)=A(IXJX)-(A(IXJ)*A(JJX))
65    B(IX)=B(IX)-(B(J)*A(IXJ))
C     BACK SOLUTION
70    NY=N-1
      IT=N**2
      DO 80 J=1,NY
      IA=IT-J
      IB=N-J
      IC=N
      DO 80 K=1,J
      B(IB)=B(IB)-A(IA)*B(IC)
      IA=IA-N
80    IC=IC-1
      RETURN
      END
C
C     THIS FUNCTION SUBPROGRAM RETURNS AREA AND OTHER VARIABLES.
C
      FUNCTION AR(Y)
      AR=Y
      RETURN
      END
      FUNCTION BR(Y)
      BR=1.0
      RETURN
```

```
END
FUNCTION CI(Y)
CI=0.02000
RETURN
END
FUNCTION CN(Y)
CN=0.02000
RETURN
END
FUNCTION DAY(Y)
DAY=1.0
RETURN
END
FUNCTION DBY(Y)
DBY=0.0
RETURN
END
FUNCTION DDAY(Y)
DDAY=0.0
RETURN
END
```

```
                    STREAMFLOW ROUTING
                  *********************
 INFLOW HYDROGRAPH IS
   5.00   6.40   8.40   9.64 10.00
INFLOW HYDROGRAPH IS LOGPEARSON T
--------------------------------------
 SLOPE=   0.2100000E-0300= 10.50
--------------------------------------

     TIME= 1.00
                VALUES OF THE VARIABL
```

DISTANCE	DEPTH	DISCHARGE
0.0	5.70	13.948
2.0	5.55	13.207
4.0	5.44	12.627
6.0	5.34	12.174
8.0	5.27	11.821
10.0	5.21	11.544
12.0	5.17	11.329
14.0	5.13	11.160
16.0	5.10	11.029
18.0	5.08	10.926
20.0	5.06	10.847
22.0	5.05	10.785
24.0	5.04	10.738
26.0	5.03	10.702
28.0	5.02	10.679
30.0	5.00	10.670